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Alexander Ziock

IRRIGATION CANALS

AND OTHER

Irrigation Works,

INCLUDING

The Flow of Water in Irrigation Canals

AND

OPEN AND CLOSED CHANNELS GENERALLY,

WITH

TABLES

Simplifying and Facilitating the Application of the Formulæ of KUTTER, D'ARCY AND BAZIN,

BY

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"Hydraulic Tables based on Kutter's Formula,"
"Flow of Water in Open Channels," etc.

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PREFACE.

It is fully admitted that a work on *Irrigation Canals* is much needed in this country. Since this work has been in the printer's hands, I have received letters from prominent engineers, from all parts of the United States, who are anxiously awaiting its issue.

The work is divided into two parts. The first part treats of *Irrigation Canals and Other Irrigation Works*, and the second part of the *Flow of Water in Open and Closed Channels, generally*.

I have aimed at making the work useful, not only to the engineer engaged in active practice, but also to the engineering student. With this object in view I have arranged the articles, as well as I could judge, in the order in which they should follow each other. It is the first time, so far as I am aware, that a work on Irrigation Canals has been arranged on this plan.

In preparing the work on *Irrigation Canals*, the best authorities have been consulted and due acknowledgment is given to them throughout the work.

Over ninety per cent. of the matter in the *Flow of Water* is original. Some of it, however, has appeared before in my other publications.

In order to simplify and facilitate the application of the modern and accurate formulæ of Kutter, D'Arcy and

Bazin, I first reduced them to the Chezy form of formula:—

$$v = c \times \sqrt{r} \times \sqrt{s}$$
$$Q = a \times c \times \sqrt{r} \times \sqrt{s}$$

Then, for open channels I have constructed three tables:—

1. Tables giving the values of a, r, $\sqrt{r}$ and $a\sqrt{r}$ for a large range of channels and for several side slopes.

2. Tables giving the values of c and $c\sqrt{r}$ for different grades and different values of n.

3. Table of slopes giving also the value of $\sqrt{s}$.

Also, for circular and egg-shaped pipes, sewers and conduits, I have constructed two tables:—

1. Tables giving the values of a, r, $c\sqrt{r}$ and $ac\sqrt{r}$ for several values of n.

2. Table of slopes giving also the value of $\sqrt{s}$. This is the same as Table 3 above.

By making $\sqrt{s}$ a separate factor, the work of computation is very much simplified.

By the use of the above Tables, any problem relating to Open or Closed Channels, likely to arise in practice, can be rapidly solved. A great saving of time and labor can be gained by the use of the tables.

There are thirty-seven examples relating to Open and Closed Channels, which will be of especial use to the student.

Tables 30, 31 and 32 give the velocity and discharge of a large range of open channels, and Tables 68 and 69 give the velocity and discharge of circular and egg-shaped pipes, sewers and conduits with $n = .013$.

At pages 8, 195, etc., is given the most complete collection of formulæ, old and modern, sixty-nine in number, that has ever before been published, in a single work, in the English language.

The *Flow of Water* will be useful, not only to the Irrigation Engineer, but also to the Engineer engaged on Water Supply, Sewerage, Drainage of Land and Improvement of Rivers, etc.

The Tables of Contents of Volumes 1 and 2, and the Index of Volume 1, are very full, and will enable the reader to find any subject, referred to in the books, without loss of time.

I have to acknowledge the many obligations I am under to Mr. George Spaulding, of George Spaulding & Co., printers, San Francisco, who has superintended the printing of the book, and also the plates for the illustrations. As a specimen of splendid typography, I refer the reader to the whole book, and particularly to the formulæ and tables.

P. J. FLYNN.

Los Angeles, California, January 9th, 1892.

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IRRIGATION CANALS

AND OTHER

Irrigation Works.

Article I. Canals divided into two Classes.

Canals are divided into two great classes, those for irrigation alone, and those for irrigation and navigation combined. The conditions required to develop one of the former class successfully, are:—

1st. That it should be carried at as high a level as possible, so as to have sufficient fall to irrigate the land for a considerable distance, on one or both sides of it.

2d. That it should be a running stream, so as to be fed by continuous supplies of water from the parent river, to compensate for that consumed in irrigating the lands.

The conditions of a canal for combined navigation and irrigation are, on the contrary, that it should be a still-water canal, so that navigation may be equally easy in both directions; and, as no water is consumed except by evaporation and absorption, and at points of transfer at locks, the required quantity of fresh supply is comparatively small, and it is thus most economically constructed at a low level.

Article 2. Systems of Irrigation.

In India there are four systems of irrigation in operation, each of them on a vast scale. They are Perennial canals, Inundation canals, Tanks or Reservoirs, and Wells. The inundation canals have no dam in the river at their heads; they give a supply only during floodtime, and the largest and greatest in number are situated on the river Indus.

Irrigation from wells is carried on, by bullock power and manual labor, each well watering from three to ten acres.

In America there are three systems of irrigation, Perennial canals, Reservoirs and Artesian Wells. In some instances in the Western States of America, water has been *developed* in small quantities by constructing *submerged* dams, in the beds of, and below the surface of the ground, of streams, and, in this manner, bringing to the surface, for purposes of irrigation, water that before had flowed to waste under the bed of the river. In other cases tunnels were driven *to bed rock* through the gravel beds of rivers and through hillsides, to *develop* water supplies.

Pumping is sometimes resorted to in America, but the most extensive pumping works in the world for purposes of irrigation are situated in Lower Egypt.

Article 3. American and Indian Irrigation Canals Compared.

In a paper in Volume I, of the Transactions of the Denver Society of Civil Engineers and Architects, by Mr. George G. Anderson, C. E., he describes the Irrigation canals of Colorado. This description is, in a great measure, applicable to the majority of irrigation canals in existence in America. Mr. Anderson states:—

"It was possible to design works on sound principles

without entering into too minute details at first, and it is to be feared that this has not been done. Regarded simply on the question of construction, it is too apparent that faults are numerous, alignments have been bad, grades and velocities established apparently without any consideration, and flumes, headworks, etc., constructed, of which a respectable mechanic would be ashamed. Still, bad as the conditions are, they have their value to the engineer, if nothing more than in showing the mistakes to be avoided in entering upon similar works in new countries. * * * *

"But by far the greater number of mistakes have been due, I think, to haste in the undertaking of the enterprise. Too little time was given or taken by the engineer in which to make himself thoroughly familiar with the physical conformation of the country to be supplied with water. Contracts were let for construction almost before a careful preliminary survey had been made, and the energetic contractor kept close at the heels of the locating engineer, with a consequence that a large percentage of necessarily bad alignment was made, which it is now utterly impossible or impracticable to correct. Probably the best thing that could occur to the irrigation system of northeastern Colorado to-day would be its entire blotting out from the face of the map, and reconstruction begun upon sound engineering principles."

Mr. Walter H. Graves, C. E., in a paper published by the Denver Society of Engineers in 1886, states:—

"To determine the proper form of channel, the proper grades, slopes, etc., requires the utmost skill and intelligence on the part of the engineer. Mistakes made in the construction of a canal may not appear at first, but subsequently develop themselves by spreading disaster and ruin on all sides. A thousand farmers depend-

ing on a canal for their water supply, at a critical period, when the canal is taxed to its utmost to supply their demands, some fatal defect suddenly appears, and the canal, for the time being, is rendered useless, and before repairs are completed the crops are ruined. A catastrophe of this kind would be almost irreparable, and through such a disaster financial ruin might overtake an entire community. The responsibility of the engineer is often too lightly assumed by him, and too carelessly and cheaply placed by the company."

The above descriptions will probably apply to over ninety per cent. of the irrigation canals and ditches in America. The weirs, headgates, bridges, drops and other works are usually temporary structures of wood.

Faulty as the works are, it must be admitted that they served a good purpose in aiding in the development of the country. Without them millions of acres of land would be waste that now bear profitable crops. There is a good field for Hydraulic engineering in the improvement of these old canals.

A great change for the better has of late taken place in the design and construction of Irrigation Canals in this country, and, in some new canals, works of a more permanent character than in the old canals, are now being constructed.

India has the greatest number of canals that can, in many respects, be quoted as good examples. It may be thought that Indian canals are too often referred to in the following pages, but it is well to remember that the finest examples of canal construction are to be seen there, that in length, cross-sectional dimensions, discharging capacity, number and aggregate mileage, the Indian canals are the greatest in the world, and that their structures are permanent, that is, that very little wood or other perishable material enters into their construction.

The experience gained in other countries, where irrigation has been practiced from time immemorial, is useful, especially in showing where mistakes have been made and the plans adopted to rectify them. Though the designs may not, on the whole, suit American practice, still many useful hints can be obtained from the study of the published descriptions of the works in other countries.

The *List of Irrigation Canals* given in Article 10, shows some of the vast works carried out in India.

Article 4. Diverting the Water from the River to the Land.

Irrigation by means of canals is chiefly applied to tracts of country which have been formed by the gradual deposit of alluvial matter, from rivers in a state of flood. The deposit from the inundation begins to take place at the points where the velocity of the stream is checked; and this being alongside the margin of the channel, an inundation of the country through which a river passes, will leave behind it, on each side, a stratum of silt in the form of a wedge, the thick end of which is on the river bank.

In the course of time, successive annual inundations will thus have formed a slope away from each of the banks. The width of this slope will vary according to the nature and size of the river. It may be only two hundred or three hundred yards wide, or it may extend to the distance of many miles.

The feature above described is not only to be found along the main channel of a river, but also along its branches. No very extensive tract of country has been formed by the inundation and consequent deposit from a single stream. On the contrary, it must have been the work of many.

The channels of all rivers, unless when confined by rocks, are more or less liable to change their course. By referring to a map of any delta, the reader will observe that the characteristics of the delta form, is that a river, as it approaches the sea, should split up into two or more branches or arms, which again may be subdivided into smaller ones. This is well exemplified in the delta of the Nile, a diagram of which is given in the block-plan, Figure 31.

Each branch of a delta has a tract of country within its influence, and serves to extend the amount of alluvial deposit, either by raising its banks or by extending the delta seaward.

It is a common occurrence to find dry beds of rivers in alluvial plains, possessing all the characteristics of the existing channels. In some cases, channels may be found of such capacity as to show, without doubt, that they are deserted beds of the main stream; in others, there may be indications of a partial and gradually diminishing supply having reached them, which, by successive annual deposits, has curtailed their section to such an extent as to admit of their being adopted as irrigation channels, or if left entirely in their natural state, such channels may be silted up completely, by successive deposits from flood water and by drifted sand and dust, until they are no longer perceptible, and all that is left to mark their course is a ridge of high land.

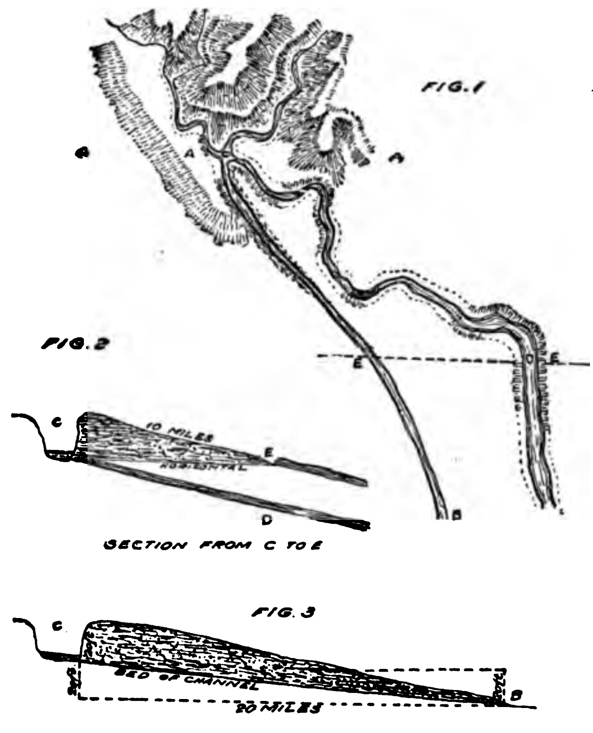
It will thus be seen that an alluvial plain is not made up of an equable deposit of alluvial matter to the right and left of the main channel of a river, but, on the contrary, by that from a number of channels, some of which may subsequently be obliterated. The fall of the country, also, instead of only following the course of the main channel, will be affected equally by all the others. Intermediate between the channels, the ground will be

low, and the line formed by the intersection of the two planes sloping away from their respective banks, will evidently indicate the course in which the drainage from those plains will tend to flow. Such lines will be found also on the extreme boundaries of a delta, receiving on one side the drainage of a portion of the delta, and on the other that of the country independent of it.

After these remarks it is time to explain that the irrigation of a tract of country is based on very simple principles. Supposing that a supply of water is required for the land near the bank of a river, which has ceased to overflow it, but which may rise to the lip of the channel, then as the country falls away from the river, it will be readily understood that a cut through the bank will give the means of irrigating the ground beyond. This may be considered the simplest form of irrigation. Again, if the surface of the river falls so considerably below the lip of the channel, as to be incapable of supplying water to the land at a distance, by means of a cut carried at right angles to the course of the river, the difficulty may be surmounted by excavating a channel in an oblique direction; for the course of a river is seldom straight for a few miles, and an artificial channel may be formed in a straight line, which will carry water to a higher level than that of the surface of the river at any point opposite to it. For every mile of its course, it thus gains something on the surface of the level of the river, and it becomes a matter of simple calculation to find how far it will have to be carried before the water issues on the surface. For example, let the plain below *B*, Figure 1, require to be irrigated from the river *C D*.

Suppose that the surface of the country from the foot hills at *A*, *A*, *A*, to *B*, falls at the rate of two feet per mile. Let the country be traversed by a river, *C D*, and

let the surface of the water in this river throughout its length be about twenty feet below its banks. If, then, a channel, *C E*, be excavated with a horizontal bed and the water at *C* raised very slightly by a weir in the river at this point, then the water from the river above *C* would flow along this channel until it reached *E*, a point at right angles to the river at *D*, whence the water might be conducted to irrigate the lower portions of the slope, *E B*.



In like manner if the bed of the channel were made to fall one foot per mile, it would at ten miles be only ten feet below the country at *E*, and at twenty miles, having gained a foot per mile, it would emerge on its surface at *B*.

When, however, the ground falls at right angles to, as well as with the course of the river, the water would come to the surface of the ground at a less distance from *C*, than 20 miles.

The case is more unfavorable, in the higher reaches of a river above the delta, where the country slopes upwards away from the river. In this case the water for the lands farthest from the river must be brought from a part of the river nearer to its source, and the excavations must be deeper; or, as will often happen, the expense bearing too high a ratio to the attainable advantage, the irrigation must be restricted to those lands which lie nearest to the source of the river, and at the lowest levels.

It is the depth of the surface of the water below the bank of the river at the head of the channel, and the relative slope of the bed of the channel and the surface of the country through which it passes, which determines the least length of the channel.

In order to obtain command of level, and in order to get on the high ground without much heavy digging, it is sometimes necessary to locate the head of the canal high up on the river's course. For this purpose it is sometimes necessary to go either to the spot at which the river finally leaves the hills to flow through the plains, or to a point not far below that spot. Moreover, at this point the water, except in freshets, is comparatively free from silt, the great enemy of canals, and the course of the river is restricted within narrow limits, so that, by dams thrown across the river bed, we can easily divert the water into our new channel.

The above considerations are so important, or rather peremptory, that they outweigh the disadvantages of the arrangement which are, indeed, very serious. For the country so close to the hills having generally an exces-

sive fall, and being, moreover, intersected by hill torrents, the carrying of the canal through such irregular ground entails serious difficulties, which require the greatest engineering skill and a large expenditure of money to overcome them.

Referring to the canal through the delta, it will be readily understood that the high ridges and the old channels, above described, indicate the most suitable alignment for a series of irrigation channels. The object would be to conduct the water from the river to the crest of such high lands, and then for the channels along them, to arrange as far as may be practicable that the excavation shall be no more than sufficient to furnish the material required for the embankments, which should retain the water at as high a level as possible, consistent with their stability. If the depth of water admitted into the head of the main channel is materially less than what is due to the river at its full height, the depth of excavation at the head will increase in proportion to the difference; and it will then be our object in order to make the cutting as inexpensive as possible, to carry the line of the channel through low ground, until the water would flow on the surface. The irrigation limit is then reached, and the channels should be continued along the highest ground that will allow of the water continuing on the same level with it or above it, as may be found most suitable for the locality. If the ground were level on both sides of the channel it would, in many cases, be indispensable to have the surface of the water above it; but on the other hand, the soil may be ill adapted for withstanding pressure, or for preventing percolation; and to avoid the occurrence of breaches it may be desirable to keep the height of the embankments within very moderate limits.

The selection of the exact spot for the head of a canal

is a task requiring much careful consideration. This subject will be again referred to in some of the following articles.

Article 5. Quantity of Water Required for Irrigation.

The source of supply for an irrigation canal having been fixed, the next point for consideration is the quantity of water required. This quantity depends upon:—

1. The maximum quantity of land requiring irrigation during the same period.
2. The duty of water in the locality irrigated by the canal.

The duty of water is the area irrigated annually by one cubic foot of water per second. This subject is discussed in the article entitled *Duty of Water.*

If, after an examination of the map of the irrigation district, we find that 96,000 acres require to be irrigated during one season, and we also find the duty of water in this district, or in a district similarly situated, to be 120 acres, then the quantity of water required to enter the head works of the canal is $\frac{96000}{120} = 800$ cubic feet per second.

A different method of estimating the quantity has been adopted in the projects for some of the Indian canals.

For the Sone Canals in India, three-quarters of a foot of water per second was estimated as sufficient for every square mile of *gross* area, but this area included land watered from existing wells, land lying fallow, village sites, roads, etc.

For the Upper Ganges Canal in India, eight cubic feet per second was allowed per lineal mile of main canal, and on the Sutlej Canal, six and seven cubic feet have been taken on the same basis.

If the canal is to be a navigable one, a certain minimum depth of water must be kept in it to float the boats

as far as the navigation extends, and this must be in excess of the quantity required for irrigation.

In India the following canals have provided for the purpose of navigation alone, in addition to the irrigation supply:—On the Sone Canals 600 cubic feet per second, on the Baree Doab Canal 130 cubic feet, and on the Ganges Canal, 400 cubic feet per second.

In fixing the area available for irrigation, all swamp land, sites of towns, roads, etc., not requiring water have to be deducted, and only the remaining area computed, which actually requires water.

Having determined the quantity of water required, the next step is to fix the dimensions and grade of the canal.

Article 6. Depth to Bed-width of Canal, and Dimensions of Canals.

The form of cross-section of a channel is determined in a great measure:—

1. By the purpose for which it is intended.

2. By the material through which it passes.

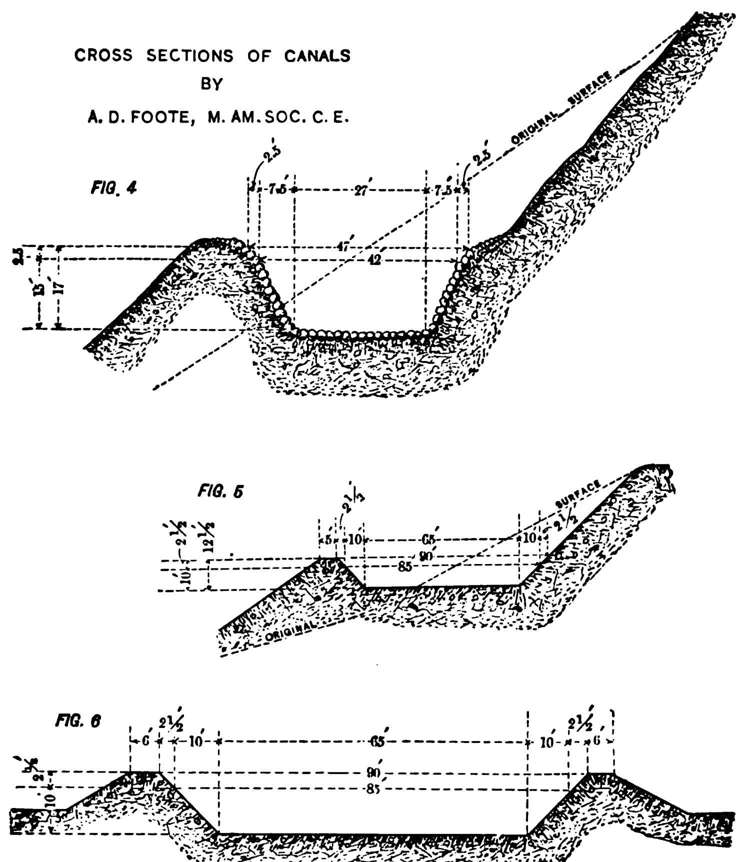
3. By the topography of the country, that is, whether it passes over a plain or along a steep hillside.

A rectangular channel having a width equal to twice the depth, has a maximum discharging capacity for the same cross-sectional area. The nearer a channel approaches this form the less will be its sectional area, for the same discharge, and, therefore, the more economical will it be.

If the object is to convey water to a certain point without expending any of it until that point is reached, and if the material cut through will bear a high velocity, then it is advisable to adopt a section having a bottom width equal to about twice or three times the depth,

and with such side slopes as may be required. All the fall available can be used so long as the velocity will not erode the bed or banks, or endanger the works.

On steep hillsides, also, this form of channel can, in some cases, be used with advantage, where the material is good, as already explained. In this case the upper



side is usually all in cut and the lower side partly in cut and partly in fill.

If, however, the channel is used to supply other minor

channels with water for irrigation, its depth should be small in proportion to its width, in order that, when the supply fluctuates, the surface of the water may be near the surface of the land to be irrigated.

For rectangular channels, constructed of masonry or concrete, the maximum discharging channel of given area is one with a bed-width equal to twice the depth.

The diagrams, figures 4 to 11, show cross-sections of some existing canals in America, India and Spain.

In the *List of Irrigation Canals*, Article 10, the proportion of depth to width can be seen by inspection. It will be noticed in the Indian canals that the proportion of depth to width is less than in European and American canals. The greater number of the Indian canals flow through sandy loam, and their mean velocity seldom exceeds three feet per second. In order to arrange for a low velocity, and also to keep the surface of the water in the canal, at all periods of ordinary supply, at such a level as to be able to irrigate the adjacent land, the depth has been made from one-tenth to one-twentieth of the width, except in the case of the Agra Canal, where it is one-seventh.

On the Western Jumna Canal, an old canal in India, the water, in the course of years, formed for itself a channel whose depth was found, by a series of trials, to be about one-thirteenth of its width. After this, the proportion of depth to width fixed on construction for the following canals in India was: on the Baree Doab Canal 1 in 15, on the Sutlej Canal 1 in 14, and on the Sone Canals 1 in 20.

A rule has been proposed to make the bottom width equal in feet, to the depth in feet plus one, squared.

Mr. T. Login, who was for many years an executive engineer on the Ganges Canal, has given the following table, showing approximately the sections and slopes,

probably best adapted for irrigation canals and water courses for Northern India.

The velocities are computed by Dwyer's formula—

$$v = 0.92\sqrt{2fr}$$

Where r = hydraulic mean depth in feet.

" f = fall in feet, of surface of water per mile.

" v = mean velocity in feet per second.

TABLE 1. Giving dimensions and grades of canals.

Cubic feet of discharge per second.	Sections.				Sections.			Side slopes of channels.
	v Mean velocity in feet per second......	Breadth of channel at bottom in feet......	Depth of water with full supply in feet..	Slope of surface of water in inches per mile..............	Breadth in feet at bottom..............	Depth of water with full supply in feet..	Slope of surface of water in inches per mile..............	
50	2.	2½	4	15	2	4½	16⅓	1 to 1
100	2.25	4½	4¾	14½	4	5¼	16	1 to 1
250	2.5	15	5	13½	13½	5½	14⅝	1 to 1
500	2.75	27½	5¼	13	25	6	14⅛	1 to 1
1000	3.	50	6	13	45	6⅜	14⅛	1 to 1
2000	3.25	77½	7	13	70	7¾	14⅛	1½ to 1
3000	3.5	95	8	12¾	85	8⅝	14	1½ to 1
4000	3.5	121½	8½	12¾	110	9⅛	14	1½ to 1
5000	3.67	147½	8½	12½	130	9⅛	13¾	1½ to 1
6000	3.75	170	8¾	12½	150	9⅜	13¾	1½ to 1

While the dimensions given in the above table are, doubtless, suitable for the locality mentioned, still the slopes assigned will not give the velocities stated in the

table. They are computed by a formula with a ***constant*** co-efficient $c = 94.5$. To prove this we have:—

$$s = \frac{f}{5280} \therefore f = 5280 \times s$$

substitute this value of f in Dwyer's formula and we have:—

$$v = 0.92 \sqrt{2 \times 5280 \times s \times r}$$

$$\therefore v = 94.5 \sqrt{rs}$$

It is now admitted that a formula with a constant co-efficient, such as Dwyer's, is suitable for only a small range of channels. It is, however, now generally accepted that Kutter's formula is applicable to a wide range of channels, and that, of all the existing formulæ, it gives the closest approximation to the actual flow of large open channels.

Assuming a value of $n = .025$ for the channels given in table 2 below, we find the corresponding velocities. These velocities show that Dwyer's formula used in computing table 1 above, gives too high a velocity for all the channels. This subject will be referred to at length in the articles on the *Flow of Water* where the application of Kutter's formula is fully discussed.

TABLE 2. Giving velocities of Channels by Kutter's formula with $n = .025$.

Breadth of channel at bottom in feet.	Depth of water with full supply in feet.	Slope of surface of water in inches per mile.	Side slopes.	Velocity in feet per second.
2½	4	15	1 to 1	1.34
4½	4¾	14½	1 to 1	1.61
15	5	13½	1 to 1	1.98
27½	5½	13	1 to 1	2.24
50	6	13	1 to 1	2.53
77½	7	13	1½ to 1	2.87
95	8	12¾	1½ to 1	3.12
121½	8½	12¾	1½ to 1	3.30
147½	8½	12½	1½ to 1	3.30
170	8¾	12½	1½ to 1	3.39

Article 7. Side Slopes.

The side slopes usually adopted, on the water side, are within the limits of ½ horizontal to 1 vertical, to 3 horizontal to 1 vertical. For most soils a flatter slope than 2 to 1 will not be required, and it is very seldom that as flat a slope as 3 to 1 is required.

The outer slopes in earthen soils may have an inclination regulated by the stability of the ground, and 1½ to 1 is most common.

As every rule has an exception, so we find that Mr. Walter H. Graves, C. E., states of the Grand River Canal system of Colorado:—

"The soil of this locality is peculiar, a sort of argillaceous adobe, that when dry resembles ashes, and when thoroughly wet becomes a slimy mud, that is almost im-

[illegible] with which [illegible] be made. In [illegible] adopted, while in the [illegible] preference, chiefly for eco- [illegible] might seem the former of [illegible] stronger construction, but an

officer of irrigation experience is able to distinguish between a rajbuha (lateral or distributary) newly made, and one that has settled down into an irrigating line. Whatever slope is adopted in construction, it is found that this cannot be advantageously maintained after the channel has been in use for some time. A distributary at the close of an irrigating season invariably assumes the following shape, except when the soil is impregnated with reh (alkali).

FIG. 7

When the time for clearance comes round, the engineer in charge, if he is wise, will not attempt to restore the original section which is opposed to the form that nature adopts. It is only waste of money to dig away the long slopes which are soon recovered with silt, while in theory it does not afford a maximum hydraulic depth which it is the great object to attain in channels with low velocities. The custom on the Ganges Canal Distributaries is to trim off the slope at $\frac{1}{2}$ to 1, as shown by thick dotted lines, *a b*, *c d*, in Figure 7; and this, in practice, is found to conform more closely to the average working section than any other. To arrive, then, approximately, at anything like the working results, the discharge tables employed by the rajbuha (distributary or lateral) designer should be based on these conditions."

The following canals have the side slopes mentioned opposite their names. More information relating to these and other canals will be found in Article 10.

TABLE 3. Giving the inner side slopes of canals in earth and sandy loam.

Ganges Canal, India	1½ (horizontal) to 1 (vertical).
Sone Canals, India	1½ (horizontal) to 1 (vertical).
Sutlej Canal, India	1 (horizontal) to 1 (vertical).
Agra Canal, India	1 (horizontal) to 1 (vertical).
Cavour Canal, Italy	1½ (horizontal) to 1 (vertical).
Henares Canal, Spain	1½ (horizontal) to 1 (vertical).
Del Norte Canal, Colorado	3 (horizontal) to 1 (vertical).
Citizens Canal, Colorado	3 (horizontal) to 1 (vertical).
Turlock Canal, California	2 (horizontal) to 1 (vertical).
Central Canal, California	2 (horizontal) to 1 (vertical).

TABLE 4. Giving the natural slopes of materials with the horizontal line.

	Degrees.		Degrees.
Gravel, average	40	Shingle	39
Dry sand	38	Rubble	45
Sand	22	Clay, well drained	45
Vegetable earth	28	Clay, wet	16
Compact earth	50		

Article 8. Grade or Slope of Bed of Canal.

The method of finding the grade of a channel of given dimensions, in order that it may have a certain velocity or discharge, is fully explained in the article on the *Flow of Water*, as well as the solution of all the other problems relating to open channels, likely to occur in practice.

The discharge of an irrigation canal is diminished in proportion to the quantity of water expended for irrigation from its head to its tail end. There are three methods by which the diminution of the discharge is regulated:—

1. By diminution of sectional area and an increase of slope, so proportioned that, though the discharge is reduced as required, still the velocity is not diminished throughout the full length of the canal channels.

2. By keeping the same sectional area and diminishing the longitudinal slopes or grades.

3. By maintaining the same grade and diminishing the sectional area.

An example of the *first method* is herewith given in detail.

Where the fall of the country is tolerably uniform, the slope of the bed of the main channel should be less than that of the branches, which again should be less than that of the minor channels and cuts. The object of this is to secure, as far as possible, a uniform velocity so that the alluvial matter held in suspension may be carried on from the head, and deposited uniformly over the lands irrigated.

There are two important reasons why the silt should be carried on to the land, the first is that the annual silt clearance from the canal may be lessened as much as possible, and the second is, that the silt, if it has fertilizing qualities, is of great benefit to the land. The benefit derived from this is fully explained in the article entitled *Fertilizing Silt*.

As to the actual fall which should be given to a main canal, of say bed width 100 feet, and depth of water 6 to 10 feet, experience shows that about 6 inches to 1 foot in a mile is ample, with a wetted border of average roughness.

The *List of Canals*, in Article 10, gives the grade of the principal Irrigation Canals in existence.

Let us now assume that a canal having a capacity of 1,700 cubic feet per second is required to irrigate a certain district. Experience on other canals in the district has shown that a mean velocity of 2.5 feet per second will prevent the deposition of silt, whilst at the same time it will not erode the bed or banks. It is therefore determined to give the canal a bed-width of 100 feet, a depth of water of 6.5 feet, and side slopes of 1 to 1. By Kutter's formula, with $n = .025$, we find that a slope of 10 inches per mile will give a velocity of 2.5 feet per second, and that therefore the discharge is 1,730.6 cubic feet per

second which is near enough to the required discharge for all practical purposes.

Let us now suppose that branches are drawn off to supply water for irrigation, and that, after these supplies are drawn off, the bed-width and depth of the channel are reduced, below the head of each branch. As the mean velocity throughout is to be maintained at what the channel had at starting, the grade of the canal will have to be increased at each dimunition of discharge. For example, at the tenth mile from the headwork an irrigation channel takes off a supply of 550 cubic feet per second. This leaves a supply of 1,150 cubic feet per second in the main canal. Arranging the dimensions required for this supply, we find that a bed width of 80 feet, depth of water of 5.5 feet, side slopes of 1 to 1, and a grade of 13 inches per mile, will, by Kutter's formula, with $n = .025$, give a discharge of 1,175.6 cubic feet per second, and a velocity of 2.50 feet per second.

These agree near enough to the required velocity and discharge for all practical requirements.

At the 10th mile a branch takes off 550 cubic feet per second.

At the 19th mile a branch takes off 350 cubic feet per second.

At the 31st mile a branch takes off 300 cubic feet per second.

At the 40th mile a branch takes off 260 cubic feet per second.

At the 54th mile a branch takes off 140 cubic feet per second.

At the 60th mile a branch takes off 50 cubic feet per second.

The channel at the tail of the canal has only 50 cubic feet per second for ten miles.

The table given below shows how the dimensions and grades of the channels are arranged to give the discharge required, which is shown in the sixth column.

It will be seen that the discharge by formula, given in column seven of the table, differs a little from the required discharge in column six, but a slight difference of this amount does not affect the work to any appreciable extent.

TABLE 5. Giving full details of channels computed by Kutter's formula with $n = .025$.

Bed, Width in Feet.	Depth in Feet.	Side Slopes	Grade per mile.	Computed Mean Velocity in feet per second.	Required Discharge in Cubic Feet per second.	Computed Discharge in Cubic Feet per second.
100	6.5	1 to 1	10 inches	2.50	1700	1730.6
80	5.5	"	13 inches	2.50	1150	1175.6
60	5.0	"	15 inches	2.48	800	806.0
40	4.5	"	19 inches	2.52	500	504.6
20	4.0	"	2 feet	2.43	240	233.3
10	3.0	"	4 feet	2.64	100	103.0
6	2.5	"	5 feet	2.45	50	52.0

From the length of the different reaches of the canal and the fall in feet per mile in each reach we find that in the whole distance of 70 miles the total fall is 150 feet, being an average fall of 2.2 feet, nearly, per mile. If the fall of the country did not admit of so high an average, it might be easily reduced by maintaining a greater depth in the channels and diminishing the width. A greater depth would give a greater hydraulic mean depth, and, according to the increase of the hydraulic mean depth, the slope could be diminished.

The above will be sufficient to indicate the mode in which the slope of the channel should be regulated, in order to prevent accumulations of silt. In practice, a canal is never perfectly aligned on this principle, but every endeavor should be made to adhere to it, in designing a system of irrigation works, so far as local peculiarities and other circumstances will permit.

The accumulation of silt in channels, particularly in the main channel, is not only a serious impediment to maintaining a supply of water till the crops are matured, but the clearance may be enormously expensive. Even if the silt cannot be carried on to the fields, as in a perfect canal, at least one step in advance is gained, if it is prevented from accumulating in the main channel; for the maintenance of the supply in it, is the most essential point, and if there are deposits in the branches only, it may be possible to clear them in turn, without cutting off the supply from the river. If this might not be feasible with the branches, it would be so at all events with the smaller irrigation channels; and it would not only be advantageous to throw on the slit to them, and to clear them in turn, without cutting off the supply of water from the branches, but the clearance would evidently be much less costly from them than it would be from the larger channels, because the haul would be less.

When the fall of the country is so gentle as not to allow of the fall of the channels being gradually increased from ten inches a mile it would be necessary to reduce the initial slope somewhat. A very slight reduction, would, as it affects the whole of the channel onwards, in the aggregate, amount to something considerable.

If, on the other hand, the fall of the country be too great, the initial slope may be increased, with, if necessary, a reduction in the depth of water; or, if the fall of

country is rapid at first and afterwards more gentle, the desired result may be obtained by constructing perpendicular drops at intervals.

Any change of direction causes a certain loss of velocity, and the water thrown into branches and minor channels would lose velocity in passing through head-sluices, unless they possessed the full water-way of the channel. Due allowance would have to be made for this by adding somewhat to the slope at the heads of the branches and channels. Where the water supply is drawn from a river highly charged with silt, the principle above described of the necessity of keeping up the velocity to the point of delivery of the water is very often neglected, and as a result canals silt up, causing additional expense to clear them out and a loss of irrigating capacity.

With reference to the *second method* it is sometimes advisable, when there is little silt, to give a uniform rate of fall to the canal, or at all events not to change it too often. It will be sometimes found preferable to reduce the gradient instead of diminishing the cross-sectional area in proportion to distribution of water along its course, and as the requirements for carrying the volume of water became lessened. An illustration of this is found in the Quinto Sella Canal, in Italy, which maintains a constant section for about fifteen miles of its length; and although in this distance about one-third of its waters are drawn off for irrigation, its capacity for the carriage of water is diminished solely by reduction of gradient, the slope of its channel being 1 in 1,000 at its derivation from the Cavour Canal, and which is gradually reduced to 0.3 per 1,000 at the end, according as its requirements become lessened.

An example of the *third method* is supplied by the canal of the Central Irrigation District of California, of which Mr. C. E. Grunsky is the Chief Engineer.

The main canal of this district has a constant slope of 1 in 10,000, and a constant depth of six feet, and its discharge is diminished by contracting the bed width of the canal.

Sometimes the fall of the ground, that is, the profile of the line, will determine the grade of the canal, after which the bed width and depth are fixed.

Mr. T. Login, C. E., has stated with reference to the water of the river Ganges, admitted into the head of the Ganges Canal at Hurdwar, that it is nearly free from silt from October to March; but as soon as the snow begins to melt in April, the water is highly charged with silt. This silt is carried down the canal in the hot season, that is, from April to September, and is deposited over the canal bed, to be again picked up and carried forward in the cold season, that is, from October to March, when the water becomes more pure. This information may be useful in other works, somewhat similarly situated, as to the period during which the water supply is highly charged with silt.

Article 9. Dimensions of Banks.

The top of the canal banks is generally from 6 to 10 feet in width, according to the material and depth of water, and it is seldom less than 1½ feet above the maximum level of the water. This, generally speaking, will be sufficient, as irrigation canals, from their position, are not subject to floods, and, as a rule, they do not receive much of the drainage of the country through which they pass, and for this reason, the effect of a very heavy rainfall would be imperceptible.

A roadway is sometimes made on one or both banks, and, in this case, this determines the top width of the banks.

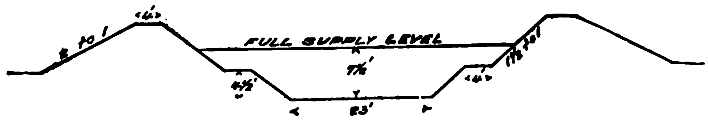
The top width of the bank is made level, or slightly

lower on the side furthest from the canal, to allow the rain waters to run off in that direction as, were the contrary the case, during storms a considerable quantity of earth, especially in light soils, might be washed into the canal.

In some cases the top of the bank is made a segment of a circle with a slight rise in the center.

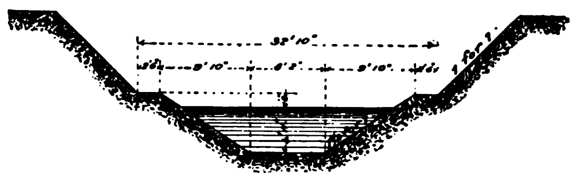
CROSS-SECTIONS

FIG. 8



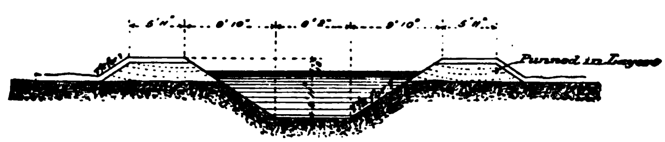
NIRA CANAL-INDIA

FIG. 9



HENARES CANAL-SPAIN

FIG 10



HENARES CANAL-SPAIN

When the channel is partly in cut and partly in fill, a berm of from 2 to 6 feet in width is left between the top

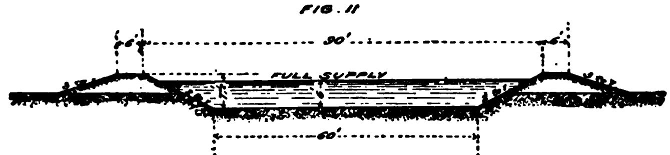
of the bank in cut and the bottom of the fill, and usually the fill has a flatter slope than the cut.

In deep cutting, where the excavated material is run to waste, a berm of at least 10 feet in width should be left between the top of the cut and the bottom of the slope of the waste bank, and the waste bank should be dressed up uniformly along the line.

In side-hill ground an open drain should be made on the high ground above the canal, and the intercepted drainage water carried to the nearest water-course.

In some of the large canals in Colorado, the bed has a slope from the sides to the center of from 1 to 3 feet, and this is called the *sub-grade*. It is said to have a tendency to keep the current in the center of the channel.

FIG. 11



CROSS SECTION OF CANAL. CENTRAL DISTRICT, CALIFORNIA

Another peculiarity of some canals in Colorado, is mentioned by Captain E. L. Berthoud, already quoted. He states:—

"When the open cutting of a ditch follows around a mountain slope, I find that the transverse 'slope' of the ditch bottom should be 0.40 to 0.70 of a foot lower than the 'bank side' of the ditch, thus throwing the wearing force of the current near the mountain side, and largely diminishing the tendency in bends to cut the 'bank' opposite the slope of cutting.

"This deepening of the transverse slope, practically in a bend, has the same effect as the elevation of the outer rail in a railroad curve."

In silt carrying channels, this lowering of the bed at sub-grade and at the inner slope in sidelong ground, seems of doubtful utility. It is likely that, in time, these channels would make a working section, in which the low parts mentioned would not be very apparent nor be very likely to have any very marked effect on the velocity at these parts.

Article 10. A List of Irrigation Canals, Giving Dimensions, Grades, Etc.

The following list gives some details of irrigation canals in the principal irrigating countries of the world. These details refer to the greatest discharging part of the canals mentioned, that is, the reach immediately below the head works.

The dimensions of the main canal only are mentioned. The laterals or distributaries are not included. For instance, the length of the Ganges Canal is given as 456 miles. This is the length of the main canal alone. It has in addition 2,599 miles of distributaries or laterals and 895 miles of escapes and drainage channels, which makes its total length of drainage channel 3,950 miles.

Again, the Sutlej, also known as the Sirhind Canal, has, including all its channels, a length of 4,950 miles, but of this only 503 miles, the length of the main canal, is given in the list. There are thousands of miles of irrigation canals, in the different countries mentioned, not included in the above list. The Inundation canals of the single province of Sind in India are over 5,000 miles in length.

The mean velocity of the canals varies from 2 feet per second upwards to 7 feet, and the side slopes from 1 to 1 to 4 to 1. As a rule, however, the side slopes of irrigation canals, when first constructed, vary from 1 to 1 to

2 to 1, but, it is generally found, that after being in use for some time, and exposed to the action of the water, they become steeper than they were originally constructed. The information about some of these canals differs considerably. For instance, the discharge of the Upper Ganges Canal, and the Lower Ganges Canal, has been stated by some authorities to be 5,100 cubic feet per second and by others as high as 7,000 cubic feet per second. The best available information has, however, been taken with reference to the canals included in the list.

TABLE 6. Giving a list of Irrigation Canals.

NAME OF CANAL.	COUNTRY.	Length in miles....	Bottom width in feet..........	Depth in feet......	Slope	Discharge in cubic feet per second...
Upper Ganges.......	India	456	170	10	1 in 4224	6000
Lower Ganges........	"	531	216	8	1 in 10560	6500
Western Jumna......	"	433				2372
Eastern Jumna.......	"	130				1068
Baree Doab	"	466	120	5.5		2500
Sutlej or Sirhind.....	"	503	190	6	1 in 4800	3500
Agra	"	137	70	10	1 in 10560	1100
Sone, Western.......	"	125	180	9	1 in 10560	4500
Sone, Eastern	"	170	180	9	1 in 10560	4500
Soonkasela..........	"	190	90	8	1 in 3520	3000
Ibrahimia	Egypt.	170	113	Lois 6¼	1 in 16600	
Main Delta (Flood)...	"		174	20	1 in 15000	10846
Main Delta (Summer).	"		174	10	1 in 12000	3943
Sirsawiah (Flood).....	"		20	17	1 in 20633	1138
Nagar "	"		20	10	1 in 14000	906
Sahel "	"		20	12	1 in 25641	981
Subk "	"		13	6	1 in 20000	114
Grand Canal of Ticino	Italy	31			1 in 1860	1851
Cavour	"	53	131	11	1 in 2000	3250
Ivrea	"	92	27.7			700
Cigliano.............	"	102	53			1760
Rotto.......	"	84				600
Muzza	"					2175
Martesana..	"					738
Henares.............	Spain..........	28	8.23	4.9	1 in 3067	177
Isabella II...........	"	50				89
The Royal Jucar.....	"	25				911

TABLE 6.—Continued.

NAME OF CANAL.	COUNTRY.	Length in miles....	Bottom width in feet..........	Depth in feet......	Slope	Discharge in cubic feet per second....
Marseilles............	France	52	9.84	7.87	1 in 3333	424
Ourcq................	"		11.48	4.92	1 in 9470	
Crappone...........	"	33	26	6.5		500
Verdon.............	"	51			1 in 5000	212
Alpines..............	"					480
St. Julien..........	"	18			1 in 3333	165
Carpentaras..........	"		33		1 in 4000	212
Del Norte............	Colorado, U.S.A.	50	65	5.5	1 in 660	2400
Citizens.............	"	45	40	5.5	1 in 1760	1000
Uncompahgre........	"	32	24		1 in 1560	725
Fort Morgan........	"	28	30	3.5	1 in 3300	340
Larimer.............	"	45	30	7.5		720
North Poudre........	"	30	20	4.0	1 in 2640	450
Empire..............	"	32	60	5.5		1400
Grand River.........	"		35	5.0	1 in 2880	
High Line...........	"	70	40	7	1 in 3000	1184
Central District	California	60	60	6	1 in 10000	720
Merced.............	"	8	70	10	1 in 5280	3400
San Joaquin and King's River.......	"	39	55	4	1 in 5280	
Seventy-Six..........	"		100	4	1 in 3520	
Calloway............	"	32	80	3.5	1 in 6600	700
Turlock	"	80	20	10	1 in 666	1500
Idaho Mining and Irrigation Co.'s......	Idaho.	75	45	10	1 in 2640	2585
Idaho Canal Co.'s....	"	43	40	4	1 in 3520	
Eagle Rock and Willow Creek..........	"	50	30	3	1 in 880	
Phyllis	"	54	12	5	1 in 2640	250
Arizona..............	Arizona........	41	36	7.5	1 in 2640	1000

Article 11. The Surface Slope of Rivers.

TABLE 7. Giving the surface slopes of rivers through the plains.

Name of River.	S	Fall in inches per mile.	Name of River.	S	Fall in inches per mile.
Mississippi above Vicksburg, Miss.	0.000050	3	Neva	0.000014	9
Bayou Plaquemine.	0.000170	11	Rhine, in Holland..	0.000150	9½
Bayou Latorische..	0.000040	2½	Seine, at Paris......	0.000137	8½
Ohio, Pt. Pleasant.	0.000093	6	Seine, at Poissy....	0.000070	4½
Tiber, at Rome....	0.000130	8	Saone, at Raconnay.	0.000040	2½
Newka	0.000015	9½	Haine	0.000100	6½

Article 12. Safe Mean Velocities.

Having determined the quantity of water, and fixed the proportion of depth to width, and a minimum for both, and, if the canal is to be navigable, this minimum is to be fixed chiefly with reference to navigation facilities. After this there still remains a very important question to be determined before we can devise the section for our channel, that is, the *slope of the bed*, on which the velocity depends.

If this slope is too great, the bed of the canal will be torn up, and the foundations of all bridges, drops and other works, will be endangered. The canal bed will be cut down and retrogression of levels take place, until the velocity of the water has adjusted itself to the cohesion of the material through which it flows. Also, the level of the surface of the water in the canal will be lowered and, furthermore, the difficulties of navigation against the stream will be largely increased.

If, on the other hand, the slope is too small, a larger

section of channel will be required to discharge a given quantity of water, and many additional works will be required in the shape of drops, locks, etc. There will also be danger of silt being deposited in the bed, or of the canal being choked by the growth of aquatic plants.

In order to provide somewhat against the deposition of silt, it is of the utmost importance that the grades and dimensions of the channels should be so arranged that the velocity of the water may not diminish from the time it enters the head of the canal until it is deposited on the land to be irrigated.

The removal of silt, deposited by a low velocity, has caused a great deal of trouble and expense on some of the Indian canals. On the Sone Canals dredging has to be resorted to in order to keep the channels clear. In 1882 the Arrah and Buxar canals were closed to allow the silt deposited below the head-sluices at Dehri to be cleared out by manual labor. It was estimated that about forty thousand dollars would be expended in clearing out some five or six miles of canal below the headworks.

On the Egyptian canals the necessity for the annual clearance of silt from the irrigation canals, has been one of the greatest evils of the irrigation system in that country.

It is, therefore, of the utmost importance, to keep clear of both extremes; but it is not always easy to do so, and in general a compromise has to be made. Moreover as the velocity increases rapidly with the depth, it is evident that a slope of bed which might be a very proper one for water of a certain depth, would be too great if it were necessary to increase that depth so as to throw an extra supply into the canal.

The minimum mean velocity required to prevent the deposit of silt or the growth of aquatic plants is, in Northern India, taken at 1½ feet per second.

It is stated that, in America, a higher velocity is required for this purpose, and it varies from 2 to 3½ feet per second.

In Spain it has been observed that a velocity of from 2 to 2¼ feet per second prevents the growth of weeds, but does not scour the channel.

In the Inundation Canals of Sind, in India, a province watered by the Indus, it is found that with a velocity of over 2 feet per second the silt is carried on to the fields, and, as a rule, the sand is deposited in the canal and this sand has to be cleared out every year, in order to keep the canals in working order.

In Egypt, when the velocity is less than 1.8 feet per second, silt is deposited and an immense quantity of it has to be removed every year from the irrigation canals there. A velocity of over two feet per second, however, in August and September, when the Nile water is much charged with slime, prevents deposits, not only of slime but even of sand. During summer there is no silt as the water is clear.

Having fixed the minimum velocity and depth of channel, the required slope can be computed as explained in the examples of the application of the Tables relating to the *Flow of Water*.

The maximum mean velocity is not, however, so easily fixed. It must, in the first place, vary with the nature of the soil of the bed. A stony bed will stand a very considerable velocity, while a sandy bed will be disturbed if the velocity exceeds 3 feet per second. Some gravel beds will bear a high velocity. Good loam with not too much sand will bear a velocity of 4 feet per second.

It is better to give too great than too small a velocity, as, in the former case, measures can be adopted to protect the side slopes, or falls can be made in the canal and the longitudinal slope, and, therefore, the velocity

reduced. In the latter case the deposition of silt will necessitate an annual clearance of the canal, at great expense, and the loss of ground along the canal banks on which to deposit the spoil.

The Cavour Canal in Italy, over a gravel bed, has a velocity of about 5 feet per second.

The Naviglio Grande and the Martesana canals in Italy, which are both used largely for irrigation, have steep slopes, and their mean velocities are not less than from 5 to 6 feet per second in their upper portions.

On the Arles branch of the Crappone Canal in the South of France, the mean velocity is 5.3 feet per second, and on the Istres branch of the same canal the mean velocity is 6.6 feet per second.

The mean velocity of the Baree Doab Canal in India, when carrying its supply, 3,000 cubic feet per second, is about 5 feet per second over a gravel bed.

The Del Norte Canal in Colorado, has a discharge of 2,400 cubic feet per second. At its head its bed-width is 65 feet, depth of water 5½ feet, and side slopes 3 to 1, therefore its velocity must be over 5 feet per second, but as the channel is excavated almost entirely from a coarse gravel, drift and rock, no danger is anticipated from the erosive force of the current.

Again, if the navigation requirements are to be considered, the maximum velocity at which a boat can be navigated against the current at a profit, is evidently a very intricate problem, depending on such varying data as the moving power employed, whether steam, animals or man; the description of boat, value of the cargo, etc. If the saving thus effected on the total traffic annually conveyed would defray the interest of the increased capital required for the proposed reduction of slope, then it would doubtless be desirable to make that reduction, looking at the question from that point of view only.

But there is a limit to the reduction of slope beyond a certain minimum, as explained above, owing to the paramount necessity of preventing the deposit of silt in the canal channel, and though, with canals carrying from 2,000 to 5,000 cubic feet per second, 6 inches per mile may be taken as the minimum limit, which would, under ordinary circumstances, interfere seriously with navigation; still it must depend of course on the fall of the country and the nature of the soil, and so difficult is it often found to combine the requirements of the two purposes, irrigation and navigation, that it has been seriously proposed to provide for the latter by separate still-water channels, made alongside of the running canal itself.

In the irrigation districts in this country there are numerous instances of canals and ditches with too great a slope. In other cases the woodwork of the drops has been washed away and not replaced, and by retrogression of levels the fall at the drops has been added to the original slope of bed, and in this way a velocity sufficient to erode the bed and banks has been produced. The deep channeling has lowered the surface of the water to such an extent that the distributing channels have to be deepened at their offtake in order to obtain their supply.

In some of the Indian canals, including the Upper Ganges and Jumna canals, the slope, and consequently velocity was too great, and dangerous erosion took place. To prevent dangerous channeling, expensive repairs and protective works had to be undertaken, with the additional loss of the canal for irrigation during the period that this work was going on. In computing the slope for the Ganges Canal, Sir Proby Cantley used the formula of Dubuat. This formula was often used in canal work at this time, but it is now known to be unre-

liable, especially for large canals. After the admission of water into the canal it was found that the velocity exceeded that originally contemplated. It was dangerous to the works and a great hindrance to navigation. Some years after the canal was in operation Major J. Crofton, R. E., was appointed to prepare plans for remodeling the canal. He made observations on the velocity in the canal, and also collected data on the same subject, which is herewith given from his report.

In a portion of the channel of the Eastern Jumna Canal lying in the old bed of the Muskurra torrent, where the current seemed perfectly adjusted to a *light, sandy soil,* Major Brownlow, the Superintendent of the canal, found the velocities of the surface to be from 2.38 to 2.28 feet per second, or mean velocities (multiplying by 0.81), 1.928 to 1.847 feet per second.

In the lower district of the same canal, near Barote and Deola, the maximum surface velocities, with a fair supply, were found to be 2.817 and 2.507 feet per second, or mean velocity of 2.282 and 2.03 feet per second. Silt is constantly being deposited here.

About 1,000 feet below the Ghoona Falls, on the same canal, in *very* sandy soil, with nearly a full supply of water, the maximum surface velocity was 3.077 feet per second; no erosion from bed or banks, except when a supply, much in excess of the maximum allowed, is passing down.

Below the Nyashahur bridge on the same canal, where the soil is clay, shingle and small bowlders, Lieutenant Moncrieff, R. E., found the mean surface velocity to be 6.75 feet per second, or the mean velocity about 5.47 per second. The same officer observed the surface velocity at some distance below the Yarpoor Falls in the new center division channel of the Eastern Jumna Canal, and obtained a mean of 3.96 feet per second, or about

[illegible] channel
[illegible] current.
[illegible] water-courses of the
[illegible]
[illegible] summary [illegible]
[illegible]
[illegible] surface
[illegible]
[illegible] and this.
[illegible] was [illegible]
[illegible] the [illegible] valley.
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[illegible] channel by
[illegible] section here and
[illegible] the [illegible] aqueduct.

was 3.04 feet per second. The deepest portions of the channeling out here have been silted up.

At the 50th mile, main line, below the Jaolee falls, with present full supply in the canal, the observed mean velocity was 3.06 feet per second. Erosion from the banks has ceased here; silt on the deepened bed, soil sandy.

Above Newarree bridge, 94th mile, in a stiff clay soil, with full supply in, the observed mean velocity was 4.12 feet per second. Erosion trifling here; no silt deposit.

Observations communicated by Colonel Dyas, R. E., Director of Canals, Punjab.

On the Hansi branch of the Western Jumna Canals, silt was deposited with mean velocities of from 2 to 2.25 feet per second. The deposition of the silt, however, obviously depends on the quantity and specific gravity of the matter held in suspension by the water coming from above, and the ratio of the current velocities at different points along the channel. He states from observations on the channels of the Baree Doab Canal, that in sandy soil:—

"2.7 feet per second appears to be the highest mean velocity for non-cutting as a general rule, for there are soft places where the bed *will* go with almost any velocity; but those sorts of places can be protected."

Again he states:—

"Bad places might be scoured out with a mean velocity of 2.5 feet per second, but better soil would be deposited in place of the bad with a slightly smaller velocity than 2.5 feet; and, as the supply is not always full, there would be no fear of not getting that slightly smaller velocity very frequently. The good stuff thus deposited would not be moved again by any velocity which did not exceed 2.5 per second."

In *Neville's Hydraulics*, 0.83 to 1.17 feet per second are mentioned as the lowest mean velocities which will prevent the growth of weeds. This, however, will vary with the nature of the soil; vegetation also is much more rapid and vigorous in a tropical climate than that where Mr. Neville made his observations.

In Captain Humphrey's and Lieutenant Abbott's report on the Mississippi, 1860, it is mentioned that the alluvial soil near the mouth of the river cannot resist a mean velocity of 3 feet per second; and that in the Bayou LaFourche, the last of its outlets, which resembles an artificial channel in the regularity of its section and general direction, and the absence of eddies, etc., in the stream, the mean velocity *does not exceed* 3 feet per second, and the banks are not abraded to any perceptible extent.

From the foregoing and other observations, and taking into consideration that the higher the velocity the less the works will cost, the following may be taken as safe mean velocities with maximum supply in the (remodeled) Ganges Canal channels:—

1. In the Ganges valley above Roorkee, 3 feet per second.
2. In the sandy tract generally between Roorkee and Sirdhana, 2.7 feet per second.
3. In the very light sand, such as that met with at the Toghulpoor sandhills, not higher than 2.5 feet per second.
4. And for the channels south of Sirdhana, 3 feet per second.

On the branches the same data to be assumed according to similarity of the soil.

There are soils, as Colonel Dyas has noted, such as light quicksand, which will not stand velocities of even 1 foot or 1½ feet per second, but these are never found

to any great extent in one place; erosion there can only have a local influence, and such places can be protected at a trifling expense. It is channeling out on long lines which is to be feared.

Article 13. Mean, Surface and Bottom Velocities.

According to the formula of Bazin—

$$v=\frac{c\times v_{max}}{c+25.4}=v_{max}-25.4\sqrt{rs}$$

$$v=v_b+10.87\sqrt{rs}$$

$$\therefore\quad v_b=v-10.87\sqrt{rs}.$$ In which v=mean velocity in feet per second.

v_{max} = Maximum surface velocity in feet per second.
v_b = Bottom velocity in feet per second.
r = hydraulic mean depth in feet and
s = sine of slope.

Rankine states that in open channels, like those of rivers, the ratio of v to v is given approximately by the following formula of Prony in feet measures:—

$$v=v_{max}\left(\frac{v_{max}+7.71}{v_{max}+10.28}\right)$$

The least velocity, or that of the particles in contact with the bed, is almost as much less than the mean velocity as the greatest velocity is greater than the mean.

Rankine also states that in ordinary cases the velocities may be taken as bearing to each other nearly the proportions of 3, 4 and 5. In very slow currents they are nearly as 2, 3 and 4.

The deductions of Dubuat are that the relation of the velocity of the surface to that of the bottom is greatest when the mean velocity is least: that the ratio is wholly independent of the depth: the same velocity of surface always corresponds to the same velocity of bed. He

observed, also, that the mean velocity is a mean proportional between the velocity of the surface and that of the bottom.

As the result of his experience on rivers of the *largest* class, M. Revy arrived at the following conclusions:—

1. That, at a given inclination, surface currents are governed by depths alone, and are proportional to the latter.

2. That the current at the bottom of a river increases more rapidly than that at the surface.

3. That for the same surface current the bottom current will be greater with the greater depth.

4. That the mean current is the actual arithmetic mean between that at the surface and that at the bottom.

5. That the greatest current is always at the surface, and the smallest at the bottom; and that as the depth increases, or the surface current becomes greater, they become more equal, until, in great depths and strong currents, they practically become substantially alike.

Article 14. Mean Velocities from Maximum Surface Velocities.

Bazin has given a very useful formula for gauging channels, by means of which the mean velocity can be found from the hydraulic mean depth and the observed maximum surface velocity. For measures in feet this formula is:—

$$v = \frac{c \times v_{max}}{c + 25.4}$$

Now let $c_1 = \dfrac{c}{c + 25.4}$ and

$$v = c_1 \times v_{max}$$

The following table will be found of great service in saving time, when using this formula:—

$$v = c_1 \times v_{max}$$

TABLE 8. Giving values of c_1.

Hydraulic mean depth in feet r.	Value of c_1.			
	For very even surfaces, fine plastered sides and bed, planed planks, etc:—	For even surfaces, such as cut stone, brickwork, unplaned planking, mortar, etc.:—	For slightly uneven surfaces, such as rubble masonry:—	For uneven surfaces, such as earth:—
0.5	.84	.81	.74	.58
0.75	.84	.82	.76	.63
1.0	.85	.82	.77	.65
1.5	.85	.82	.78	.69
2.0	.85	.83	.79	.71
2.5	.85	.83	.79	.72
3.0	.85	.83	.80	.73
3.5	.85	.83	.80	.74
4.0	.85	.83	.81	.75
5.0	.85	.83	.81	.76
6.0	.85	.84	.81	.77
7.0	.85	.84	.81	.78
8.0	.85	.84	.81	.78
9.0	.85	.84	.82	.78
10.	.85	.84	.82	.78
11.	.85	.84	.82	.78
12.	.85	.84	.82	.79
13.	.85	.84	.82	.79
14.	.85	.84	.82	.79
15.	.85	.84	.82	.79
16.	.85	.84	.82	.79
17.	.85	.84	.82	.79
18.	.85	.84	.82	.79
19.	.85	.84	.82	.79
20.	.85	.84	.82	.80

Article 15. Destructive Velocities.

Kutter (translation by Jackson) states:—

"The maximum velocities determined by Dubuat, as suitable to channels in various descriptions of soil, are taken from Morin's 'Aide Memoire de Mécanique Pratique', page 63, 1864. The first column in the following table gives the *safe bottom velocity*, and the second the mean velocity of the cross-section; the formula by which these are calculated is:—

$$v = v_b + 10.87 \sqrt{rs}$$

TABLE 9. Giving safe bottom and mean velocities in channels.

Material of Channel.	Safe Bottom velocity v_b, in feet per second.	Mean velocity v, in feet per second.
Soft brown earth	0.249	0.328
Soft loam	0.499	0.656
Sand	1.000	1.312
Gravel	1.998	2.625
Pebbles	2.999	3.938
Broken stone, flint	4.003	5.579
Conglomerate, soft slate	4.988	6.564
Stratified rock	6.006	8.204
Hard rock	10.009	13.127

"We (Ganguillet and Kutter) are unable, for want of observations, to judge how far these figures are trustworthy. The inclinations certainly have no influence in this case, as the corresponding velocities are mutually interdependent, but the variation of the depth of water is most probably of consequence, and in shallower depths the soil of the bottom is possibly less easily and rapidly damaged than in greater depths, under similar conditions of soil and of inclination. Yet this effect is not very large, while that of the actual velocity of the water is of the highest importance. Hence, it appears that these figures may be assumed to be rather disproportionately small than too large, and we therefore recommend them more confidently."

Mr. John Neville, in his hydraulic tables, states that for the materials given in the following table the *mean velocity* per second should not exceed—

0.42 feet in soft alluvial deposits.

0.67 feet in clayey beds.
1.0 feet in sandy and silty beds.
2.0 feet in gravelly earth.
3.0 feet in strong gravelly shingle.
4.0 feet in shingly.
5.0 feet in shingly and rocky.
6.67 feet and upwards in rocky and shingly.

The beds of rivers protected by aquatic plants, however, bear higher velocities than this table would assign, up to 2 feet per second.

Water flowing at a high velocity and carrying large quantities of silt, sand and gravel is very destructive to channels, even when constructed of the best masonry.

The Deyrah Doon water-courses in India had channels of sections varying from 5 x 2 feet to 10 x 4 feet, and with slopes varying from 50 feet to 80 feet per mile. They were almost all of masonry, and had numerous masonry falls from 5 to 6 feet in depth on them, and they passed over numerous and long aqueducts. The following table shows the mean velocity in these channels, computed by Kutter's formula with $n = .015$.

TABLE 10. Giving dimensions, grades and velocities of masonry channels.

Dimensions of Channel.		Slope in feet per mile.	Velocity in feet per second.
Width in Feet.	Depth in Feet.		
5	2	50	10.3
5	2	80	13.0
10	4	50	16.5
10	4	80	20.9

Before measures were taken to protect the channels the water rushed down along their course with a tre-

mendous velocity, and carrying large quantities of sand and gravel, and the abrasion injured all the masonry works on the line. The silt-laden water acted even more injuriously, as, impelled by the great velocity, it cut into the masonry with an action like that of emery powder. Even to a bed laid with large bowlders, great damage was caused: the mortar joints were washed out, the bowlders lifted out of their places and then rolled along the bed to add to the mischief. But it is to brickwork that the greatest damage was done. In fact, it requires but time to make all brickwork disappear entirely in the presence of such action. In some of the old canals there was a flooring of brick on edge over the arches of the aqueducts. On one of these aqueducts not only was the foot in depth of the brick floor entirely cut through, but deep ruts were formed in the arch itself. But it was on the *falls*, which were all formerly built after the ogee pattern, and of brick, that the damage was greatest, as might be expected. Their surfaces were cut into deep striæ, and they were in constant need of repairs, which were difficult to execute.

It was, therefore, important to keep the silt out, and this was done by building silt traps on the line of the canal.*

Except in storm sewers, which flow for only a short period every year, the mean velocity in sewers is usually kept below five feet per second.

Colonel Medley, R. E., had considerable opportunities of observing the abrading power of silt-laden water on the Ganges Canal, India; and in the "Roorkee Treatise on Civil Engineering" he writes thus:—

"Brickwork should not be used in contact with cur-

* Mr. R. E. Forrest, in the first volume of the Professional Papers on Indian Engineering.

rents with such high velocities (15 feet per second). Even the very best brickwork cannot stand the wear and tear for any length of time, and stone should be used for all surfaces in contact with velocities exceeding, say, 10 feet per second."

Article 16. Velocity Increases with Increase of Depth of Channel.

The following table is given in order to show that, in the channels usually adopted for irrigation, the velocity increases with the increase of depth. The channel 50 feet wide will, with a depth of two feet, deposit silt; at a velocity of about 1.6 feet per second, but with a depth of five feet and a velocity of about 2.9 feet per second, it will keep itself clear of deposit.

N =.0275. Side slopes 1 to 1.

TABLE 11. Giving dimensions, grades and velocities of channels.

Depth in Feet.	Bed Width 10 Feet.	Bed Width 50 Feet.	Bed Width 100 Feet.
	Slope 1 in 1000.	Slope 1 in 2500.	Slope 1 in 5000.
	Velocity in Feet per Second.	Velocity in Feet per Second.	Velocity in Feet per Second.
2.	2.173	1.582	1.136
3.	2.756	2.084	1.520
3.5	3.002	2.302	1.692
4.	3.224	2.511	1.856
4.5	3.440	2.701	2.007
5.	3.634	2.878	2.150

Article 17. Abrading and Transporting Power of Water.

Professor J. LeConte, in his "Elements of Geology," states:—

"The *erosive* power of water, or its power of overcoming cohesion, varies as the square of the velocity of the current.

"The *transporting* power of a current varies as the sixth power of the velocity. * * * If the velocity, therefore, be increased ten times, the transporting power is increased 1,000,000 times. A current running three feet per second, or about two miles per hour, will move fragments of stone of the size of a hen's egg, or about three ounces weight. It follows from the above law that a current of ten miles an hour will bear fragments of one and a half tons, and a torrent of twenty miles an hour will carry fragments of 100 tons. We can thus easily understand the destructive effects of mountain torrents when swollen by floods.

"The *transporting* power of water must not be confounded with its *erosive* power. The resistance to be overcome in the one case is *weight*, in the other, cohesion; the latter varies as the *square;* the former as the sixth power of the velocity.

"In many cases of removal of slightly cohering material, the resistance is a mixture of these two resistances, and the power of removing material will vary at some rate between v^2 and v^6."

Silt, sand, gravel and stones lose as much weight in water as a volume of water having an equal cubic content, which is generally about equal to half their weight in air. They are, therefore, easily moved, but, with the exception of silt, their velocity is less than that of the current, and the nearer their specific gravity approaches that of water the nearer their velocity approaches that of the current.

The English Astronomer Royal, in a discussion at the Institution of Civil Engineers, said that the formula for the transporting power of water, was the only instance in physical science, with which he was acquainted, in which the sixth power came really into application.

Mr. T. Login, C. E., states as the result of his observations for several years, on the Ganges Canal and other channels, that the abrading and transporting power of water increases in some proportion as the velocity increases, but decreases as the depth decreases.

Umpfenback gives the size of materials that will be moved in the botton of small streams, at the following figures:—

TABLE 12. Giving the transporting power of water.

Surface Velocity in Metres.	Gravel, Diameter in Metres.	Surface Velocity in Feet.	Gravel, Diameter in Feet.
0.942	0.026	3.091	0.085
1.569	0.052	5.148	0.170
	Cubic Metres.		Cubic Feet.
2.197	0.00515	7.208	0.182
3.138	0.209	10.296	0.738
4.708	0.618	15.447	21.826

Chief Engineer Sainjon made observations in the River Loire in France, with the following results:—

Velocity of feet per second,	1.64	3.28	4.92	6.56
Diameter of stone in feet,	0.034	0.134	0.325	0.56

In order to protect the foundations of the Ravi bridge, in India, 15-inch concrete cubes (1.56 cubic feet), were deposited around the piers. It was noted in one case, that with a velocity of probably not less than 10 feet a

second, the blocks were moved from a sandy bottom on to a level brick floor protecting the bridge. Although exposed to a more violent current they were not moved off the flooring. This evidence is somewhat in proof of Smeaton's experience, that quarry stones of about half a cubic foot, were not much deranged by a velocity of 11 feet per second, although the soil was washed from under them.

Experiments made by Mr. T. E. Blackwell, C. E., for the British Government, in the plan of the Main Drainage of London, show very clearly that the specific gravity of materials has a marked effect upon the mean velocities necessary to move bodies.

For example, coal of a specific gravity of 1.26, commenced to move in a current of from 1.25 to 1.50 feet per second.

A second sample of coal, of specific gravity 1.33, did not commence to move until the velocity was 1.50 to 1.75 feet per second.

A brickbat of specific gravity 2.0, and chalk of specific gravity 2.05, required a velocity of 1.75 to 2 feet per second to start them.

Oolite stone, specific gravity 2.17; brickbat, 2.12; chalk, specific gravity 2.0; broken granite, specific gravity 2.66, required a velocity of 2.0 to 2.25 feet per second to start them.

Chalk, specific gravity 2.17; brickbats, specific gravity 2.18; limestone, specific gravity 1.46, required a velocity of from 2.25 to 2.50 feet per second to start them.

Oolite stone, specific gravity 2.32; flints, specific gravity 2.66; limestone, specific gravity 3.00, required a velocity of 2.5 to 2.75 to start them.

It was shown in these experiments that after the start of the materials with the current, in no case did the materials to be transported travel at the same rate as the

stream, but in every case their progress was considerably less, as a rule, often more than 50 per cent. less than the velocity of the current.

Mr. Baldwin Latham, C. E., in the course of his experience in sewerage matters, has found that in order to prevent deposits of sewage silt in small sewers or drains, such as those from 6 inches to 9 inches diameter, a mean velocity of not less than 3 feet per second should be produced. Sewers from 12 to 24 inches diameter should have a velocity of not less than 2½ feet per second, and in sewers of larger dimensions in no case should the velocity be less than 2 feet per second.

Sir John Leslie gives the formula:—

$v = 4\sqrt{a}$ for finding the velocity required to move rounded stones or shingle, in which

$v =$ velocity of water in miles per hour, and

$a =$ the length of the edge of a stone if a cube in feet, or the mean diameter if a rounder stone or bowlder, also in feet.

This formula takes no note of specific gravity. Chailly has supplied this omission, and he has derived the following formula, which is just sufficient to set bodies in motion:—

$v = 5.67\sqrt{ag}$, in which

$a =$ average diameter of the body to be moved in feet,

$g =$ its specific gravity, and

$v =$ velocity in feet per second.

Experience on the irrigation canals in Northern India, where rapids are in use, has proved that a bowlder rapid, with a flooring composed of bowlders not less than eighty pounds in weight each, well packed *on end*, and at a slope of 1 in 15, will *not* stand a mean velocity of 17.4 feet per second.

Article 18. On Keeping Irrigation Canals clear of Silt.

By R. B. Buckley, C. E.

(Extracted from Proceedings of the Institution of Civil Engineers, Volume LVIII.)

There are four methods by which it is possible to exclude more then a desirable proportion of silt from entering an irrigation system:—

1. By works in the river, which will clear the water before it enters the canal.

2. By so constructing the head-sluice of the canal that only water bearing the desired proportion of silt is admitted.

3. By constructing a depositing basin near the head of, and in the canal itself, to be cleared either by dredging or by hand labor; or, what is practically the same thing, by making two supply canals from the river to the canal, one to be used while the other is being cleansed.

4. By constructing a double row of sluices, with a settling tank between, so arranged that the water is drawn off from the lower row carrying the desired amount of silt, and so designed that the deposit in the tank can be flushed back again into the river.

These systems are, of course, applicable under different circumstances. The first can be rarely used, and only when the local conditions are suitable. As, for example, when the bed of an inundation canal is perhaps 8 feet or 10 feet above the level of the bed of the river, and which canal is therefore only supplied when the river is in flood. In such a case, if a position for the head of the canal can be selected behind an island covered with brushwood, the top of which is perhaps a little below, or even slightly above the high flood level, it may be well worth the cost to make an artificial con-

nection between the head of the island and the main land, so that all the water entering the canal will first flow through the bay, found between the island and the main land, entering that bay from below. The velocity of water in the bay will thus be diminished; the water will deposit silt in the bay instead of carrying it into the canal; and if the bay be a large one the canal may work for many years without its bed silting up.

The same principles can be employed on large irrigation schemes, by altering the methods now generally adopted on these works. The almost invariable arrangement is that the weir which stretches across the river, at a height of from 8 feet to 15 feet above the bed, is cut by two sets of under-sluices, which are purposely set as close as possible to the head-sluices of the canal immediately above the weir; the floors of the head-sluices and of the under-sluices being at the same level. The under-sluices are placed in this position so that silt may not accumulate in front of the entrances to the canal, and thus impede the free entrance of boats to the lock, and of water to the canal. This object is attained by opening the under-sluices during floods, thus drawing down a rapid stream immediately in front of the opening to the canals, which scours the channel and removes any deposit that may have accumulated. At the same time that the action of the under-sluices clears the approaches to the canal, it causes the canal to be more deeply silted, for the higher velocity produced by the scour of the under-sluices removes an extra quantity of silt from the bed of the river, and it is from this rapid and silt-bearing stream, impinging directly on the head-sluices, that the canal is supplied. But if the weir were constructed with a double set of under-sluices at each end, one set being in the line of the weir, and about 200 feet from the river bank, and the other set some dis-

tance lower down the river, but connected to the upper set by a flank wall parallel to the river bank, and if the off-take of the canal were placed immediately above the lower set, the stream flowing to the upper set would not pass in front of the off-take to the canal. The silt-bearing water would pass through the upper set of under-sluices with full velocity, while that portion of the river destined for the canal would have its velocity checked, immediately opposite the flank wall, and would deposit its silt to a great extent before it reached the head-sluices of the canal. To sweep away the silt, which would be deposited between the weir and the head-sluices, it would be necessary to close the upper under-sluices and to work the lower ones. This plan would be rendered most effective by closing the head of the canal for a few hours every week, while the lower under-sluices were opened, so that the channel might be kept clean without allowing any silt-bearing water to have access to the canal.

In almost all cases the head sluice of a canal is formed by rows of single shutters, sliding in verti grooves, so that water is always first admitted to the canal from below the shutters, that is, at a level of the sluice floor. If the sluices were constructed so that the water was drawn from the top instead of from the bottom of the river, much less silt would be carried into the canal. In rivers which rise moderately it is best to have a single opening in each vent, covered by three or four shutters sliding in a vertical groove; and each of these shutters should have independent opening gear. In rivers liable to floods rising 30 feet it is necessary to have in each vent of the sluices, several openings at different levels, each opening being fitted with an independent shutter, so that water can be drawn off at different levels as the flood rises or falls. This way of

dealing with the silt can at most be but partially effective, but there are some rivers, carrying a small amount of silt, to which this system may be applied with sufficient effect to render the clearance of silt from the canal unnecessary.

The *third* method is frequently adopted on Indian canals. The first half mile of the canal is excavated with a base sufficiently large to cause a great diminution of velocity; the silt is deposited during floods, and excavated when the canal is closed during the summer, or perhaps it is dredged out at a cost even more excessive than that of excavating it by hand.

The *fourth* method is peculiarly suitable for rivers with a rapid fall. It is also most desirable where a canal runs alongside of the river for some distance before branching off into the country. If this method be adopted, the channel of the first half mile or so must be of such capacity that the velocity of the water in it, when carrying the full volume required for the canal, shall not exceed that which will allow of the deposit of the matter in suspension; so that the water, when it reaches the end of this length, shall contain only that proportion of silt which the channels below are arranged to convey to the fields. At the end of this broad channel a sluice will have to be built to carry the full discharge required in the canal with little or no head upon it. The head sluice on the river bank must be designed so that, with only a moderate flood in the river, a sufficient quantity of water can be introduced into the canal to generate a velocity of three to four feet per second in the broad reach, the flushing sluices leading back from this reach to the river being arranged to discharge a corresponding quantity, or even a larger quantity of water. These sluices might be fitted with falling shutters. The largest flushing sluice should be about 150 feet to 300

feet from the head sluice, for it is about this point that the heavy sand is deposited and where the greatest scour would be required. This system is the most effective and the least expensive for large schemes. If the head sluice on the river bank be constructed on the principle of taking water from the surface of the river, instead of from below, the minimum amount of silt will enter the broad reach, and that can under conditions, be cleared away by closing the sluices at the extremity of the broad channel for a short time, and opening all the shutters of the head sluice on the river bank and the various flushing sluices.

Article 19. Fertilizing Silt.

The quality of the silt carried by water for irrigation is a matter of great importance. Whilst in some localities it is of no use to the land as a fertilizer, still, in a great number of places, it acts as a good manure.

It is well known that for ages the fertility of Egypt has been preserved by the silt-laden waters of the Nile. Every year the Nile deposits its load of rich slime on the land, and, in consequence of this, the soil retains the fertility for which it has been famous since the earliest date of history. Such muddy water furnishes not only moisture to bring the crop to perfection, but it also brings manure to the land, and thus prevents it from being exhausted. The silt annually deposited is merely manure, which is consumed in bringing the crops to maturity. This is the reason that the land has, for so many centuries, remained within reach of the Nile flood.

In Upper Egypt large depositing basins, to retain the Nile silt, have been in use from time immemorial, with great success.

Sir B. Baker, C. E., states, respecting the fertilizing properties of the Nile water:—

"1st. That the fertility of the Nile is due to the organic matter, and to the salts of potash and phosphoric acid dissolved and suspended in it.

"2d. That these constituents are most abundant in the water during the months of August, September and October, when the river is in flood; and that it is during the period of inundation that the sedimentary matter, or mud, deposited from the water, is most valuable as a fertilizing agent."

In Lower Egypt these basins are not used. The land is flooded, but the water flows off and deposits very little of its silt.

This is known as the Improved System.

Mr. W. Willcocks, C. E., in his account of Irrigation in Lower Egypt, states on this subject:—

"In Upper Egypt, where the old Pharaonic system of basin irrigation exists, every acre of land is cultivated, and pays revenue, while the soil is as rich to-day as it was thousands of years ago. In Lower Egypt, on the contrary, where the improved system of irrigation prevails, and a triple crop is gathered, one-third of the area is uncultivated, while the remaining two-thirds are incapable of paying a higher revenue than Upper Egypt. The improved system, besides, has only lasted fifty years, and yet there is a cry of deterioration of the soil and produce from one end of the country to the other, a cry which is re-echoed by English cotton-spinners. Nature wants the slime of the Nile flood to be deposited on the land; it is now forced into the sea; and though it is not necessary to go to the full extent of Napoleon's statement, that were he master of Egypt he would not allow an ounce of slime to be wasted; yet it may be stated,

without fear of contradiction, that for every one pound (five dollars) of profit resulting from the expenditure of one thousand pounds (five thousand dollars) on the improvement of the existing irrigation system, ten pounds (fifty dollars) would be the return on money spent in a partial restoration of the basin system where the lands are cultivated, and a complete restoration where the lands are not under cultivation."

In respect to irrigation, there are four kinds of water:—

First, rain; this is almost pure, and supplies nothing to the land but moisture. The land dependent upon it must be continually renewed by manure.

Second, well water. This also is quite pure, being filtered through the earth, or, what is worse, it is often injured for irrigation by being mixed with injurious minerals, especially at the end of the dry season.

Third, tank or reservoir water. This generally contains a good deal of nourishment for plants in a state of solution, which it has absorbed in the lands it has passed over, but what it has held in suspension is almost all deposited in the bed of the tank before the water is drawn off for the fields.

Fourth, river water. This water is led direct from the rivers by canals to the fields. It deposits in the channels only the coarser parts of the silt it has brought down from the higher lands and forests, much of which is only sand. A large quantity of its most fertilizing silt is, however, conveyed to the land. So complete is the effect of this fertilization, that lands so supplied continue to bear one or two grain crops for hundreds of years without other manure. Thus the district of Tanjore, in India, is believed to produce as large crops now as it did 2,000 years ago.

Different rivers are more or less fertilizing according

as they pass through different rocky strata. Thus the Kistnah River, in India, which passes through a limestone country, has a delta which was found to produce crops 50 per cent. larger than the delta of the Godavery, which passes chiefly through a granite country.

In Midnapore, in India, the rainfall is sometimes as much as ten inches in twenty-four hours, but the cultivators are not satisfied with this. In order to gain the advantage of the manure in the river water, they drain off the rain water as quickly as possible and admit the former water. Long experience has proved to them that they get better crops by irrigating with the silt-laden water of the river than by the rain water.

The water of the river Indus, in India, is preferred to well water, owing to the fertilizing silt which it contains.

The water of the river Durance, in France, has a high reputation for irrigating purposes. In addition to the sediment mechanically suspended, it also holds much valuable agricultural matter in solution, which is considered the main cause of the waters of that river being so valuable for irrigation.

Mr. Kilgour, C. E., in the Minutes of Proceedings I. C. E., vol. 27, stated:—

"The silt in suspension in the waters of the Punjab rivers in Northern India was invaluable as a manure in the district, where, owing to the scarcity of timber, the dung of the cattle was mixed with clay, sun-dried, and employed as fuel."

Mr. George Gordon, C. E., in a paper on the storage of water, published in Minutes of I. C. E., vol. 33, says:—

"Land irrigated from a river gives a better return than that under a tank by, it is said, 25 per cent. in these parts. Whether this is principally due to the

brackish quality of the water locally collected, or to the insufficient supply from tanks, the author cannot say; probably both causes contribute."

General Scott Moncrieff, R. E., states that the price paid for the water of the Po, in Italy, was three times the amount paid for the water of the Dora Baltea, the extra value of the water of the Po being due to the fact of its alluvial silt being considered highly fertilizing, while that of the Dora Baltea is rather the reverse. He also refers to the marked difference between the meadows irrigated with the silt-bearing waters of the Durance Canals in France, and those of the clear, cold Sorgues, so much so, that cultivators prefer to pay for the former ten or twelve times the price demanded for the latter.

Mr. J. H. Latham, C. E., states, as the result of his observations in the Madras Presidency, in India, that river channels are the most prized of all the sources of supply for irrigation as they are stated to give 25 per cent. more crop per acre than either wells or tanks.

Mr. Allan Wilson, C. E., refers to the great superiority of river and tank or reservoir water for irrigation purposes, as compared with well and spring water, as an argument in favor of the formation of river and tank reservoirs. He ascertained from observation, and the experience of practical authorities in India, that sugar cane watered from tanks and rivers yields a much heavier crop than land watered from wells and springs, and the molasses produced from the former realizes double the price of the latter.

Mr. Walter H. Graves, C. E., of Denver, Colorado, states that the very means of reclaiming the arid land is a constant source of its fertilization. By irrigation the pores of the most sterile soil can be filled and compacted by the infiltration of the impalpable silt, and con-

verted into a loam of prodigious fertility. Hence, as a general statement, all lands that can be reached and supplied with water for irrigation are susceptible of cultivation.

Mr. A. D. Foote, C. E., in his *Report* on the *Irrigating and Reclaiming of Certain Desert Lands* in Idaho, gives full and interesting details of crops grown on lands irrigated by silt-laden water, and shows very plainly its great value as a fertilizer. And in a discussion on irrigation at the American Society of Civil Engineers in 1887, Mr. Foote further states:—

"The fertilizing silt which swift running water usually carries is eventually nearly as valuable as the water itself. Without it irrigation in this country would soon be a failure. No land can stand continual production without enriching, and it will be many years before our Far West can afford the ordinary artificial manures. The silt with which our western rivers is loaded in the spring and summer is so valuable, that the land irrigated by it improves even unto the heaviest cropping."

Mr. E. B. Dorsey, C. E., quotes an Idaho farmer as having said:—"I would rather give two dollars an acre for muddy water than one for clear."

Mr. C. L. Stevenson, C. E., Salt Lake City, states that:—"The waters of irrigation from the mountains annually carry with them fresh fertilizing material, so that practically it costs the average Utah farmer less to keep up his ditches and apply his waters of irrigation than it does the eastern farmer to manure his land. One field near Farmington, at first producing some sixty bushels per acre, was kept in wheat for thirty years with no other fertilizer than what was brought by the waters, and there was after the second or third year a general average yield of over forty bushels to the acre."

As every rule has an exception, so we find an exception to the almost unanimous opinion as to the value of silt-laden water for irrigation.

Major J. Browne, R. E., in the Transactions of the Institution of Civil Engineers, volume 33, states as the result of his observations in the Punjab, India, that:—"He had always understood from such cultivators as he had spoken to, that crops raised from well water were of a better quality than those raised from canal water. He could not say whether it was due to the higher temperature of well water, or to any chemical difference in the water itself, but canal-raised, were, he believed, generally inferior to well-raised crops."

It is not unlikely that the cultivators through custom, as has often been found in India, adhered to their old method of well irrigation.

Article 20. Silt Carried by Rivers.

It is sometimes of importance to know not only the quality of the silt carried in suspension by a river, with reference to its utility as manure, but also its quantity. This quantity varies greatly in different rivers, and also at different stages of the flood in the same river. In August the Nile conveyed three hundred times more solids in suspension than in May, although during the former month the volume of water discharged was only ten times greater than in May, the weight of solids in suspension to the weight of water being then $\frac{1}{25000}$, whilst in August it rose to $\frac{1}{834}$. Although the volume of water discharged in August was one-quarter less than in October, the suspended sedimentary matter was three times greater. In August the weight of sediment attained its maximum of 23,100,000 tons and in October with the greatly increased discharge the sediment de-

creased to 7,600,000 tons, the proportion of sediment to water in August being $\frac{1}{674}$ and in October $\frac{1}{2640}$.

The perennial flow of the Nile was due to the magnificent lakes of Central Africa, which lay at its source; while its annual inundation was caused by the flooding of the Atbara and Blue Nile during the rainy season. These two tributaries (although almost dried up from the end of October to the beginning of May, when mountainous Abyssinia was as rainless as Egypt,) were mighty streams from the beginning of June to the end of September, and the undoubted origin of the periodical inundations, the unfailing deposits, and the wonderful fertility of Lower Egypt.

The Godavery and Mahanuddy in India have a proportion about $\frac{1}{1100}$, but this is much less than that in the Kistna and Indus, the quantity in the latter amounting to nearly $\frac{1}{250}$ of its bulk.

In the Durance in France, with a flood discharge of 210,000 cubic feet per second, the quantity of sediment mechanically suspended increases with the flow of the river. The ordinary maximum is about equal to $\frac{1}{33}$ of the water by weight. In exceptional cases, as in August, 1858, the proportion was as high as $\frac{1}{10}$ of the water by weight. In extreme low water the proportion by weight is about $\frac{1}{1000}$. The average proportion for the nine years, 1867–75, was about $\frac{1}{550}$. It is estimated that the Durance transports annually to the sea seventeen million tons of earthy matter. It is stated that in the Vistula in floods the proportion is $\frac{1}{48}$; in the Garonne in France $\frac{1}{100}$; in the Rhine in Holland $\frac{1}{100}$; and in the Po $\frac{1}{300}$. In other rivers the proportion varies from that given above as a maximum to $\frac{1}{17000}$ as a dry weather flow.

Sir Charles Hartley, C. E., has given the following

table showing the principal characteristics of four of the great rivers of the world:—

TABLE 13. Giving length, discharge, etc., of rivers.

River.	Length in miles.	Drainage area in square miles.	Annual rainfall in cubic miles.	Mean annual discharge in cubic miles.	Mean Ratio.	Mean weight of dry sediment to weight of water.
Nile	3,300	1,293,000	892.1	22.7	39.3	$\frac{1}{1900}$
Ganges......	1,680	588,000	548.8	43.2	12.7	
Mississippi .	4,190	1,244,000	673.0	132.0	5.0	$\frac{1}{1500}$
Danube	1,750	316,000	198.0	44.3	4.5	$\frac{1}{3000}$

As the central and lower parts of the Nile flowed through an exceptionally dry and sandy region, it discharged, as shown on the table, $\frac{1}{39}$ of the annual rainfall on its catchment basin, and as regarded ratio of rainfall to discharge, compared with other rivers, it was three, eight and nine times greater than the Ganges, Mississippi and Danube respectively. Again, although the Nile had about the same drainage area as the Mississippi, its annual rainfall was 30 per cent. greater, whilst its annual discharge was six times less than that of the "great Father of Waters." Compared with the Danube, the annual discharge of the latter was double that of the Nile, although the annual rainfall of the Nile basin was four and a-half times that of the Danube.

An irrigation canal drawing its supply from a river, which carries fertilizing silt in suspension, and which has sufficient velocity to carry the silt on to the land requiring irrigation, deposits an immense quantity of good manure, in a few months, in a thin film, over the land. For example: let a canal have a bed width of

60 feet, a depth of 4 feet, and side slopes 1 to 1, and a mean velocity of 2.5 feet per second. Let this canal flow for four months, or 120 days, and during this time let its supply be derived from a river which holds silt in suspension to the extent of $\frac{1}{800}$ of the bulk of water. The discharge of the canal is 640 cubic feet per second. There are 86,400 seconds in one day. We have therefore:—

$\frac{640 \times 86400 \times 120}{800 \times 27} = 307,200$ cubic yards of fertilizing silt deposited by the canal on the land in 120 days. From this a good idea can be formed of the great advantage of manurial silt in an irrigation supply.

Article 21. Improvement of Land by Silting Up, Warping or Colmatage.

Silting up of land, warping or colmatage, is here intended to signify the improvement of land before this of little, if any, use, by the deposition of silt. Warping is usually applied to the artificial silting up of land on the sea coast, in bays and estuaries, but it is here also applied to the silting up by river water of land not within the influence of the tides. Colmatage is a French word also used in works written in English to express the same thing as silting up.

When water containing fertilizing silt is not required for irrigation it can be usefully employed in making good land out of a sterile waste. There are, no doubt, numerous localities in the United States where land can be improved in this way. A description of the improvement of some land by this method is here given.*

Above Epinal in France the course of the river Moselle is well defined and regular. Below that point it

*Irrigation in Southern Europe by Lieut. C. C. Scott Moncrieff.

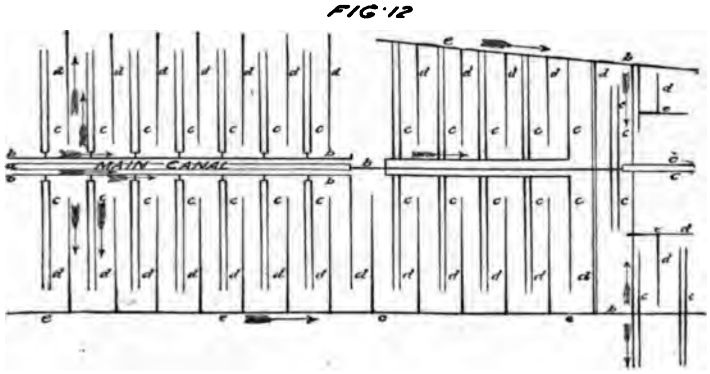
5

used to flow over a broad, gravel bed, in a number of separate streams, continually changing. A scanty crop of miserable pasturage used sometimes to spring up on the best parts of this broad channel, the rest was quite barren. This worthless strip of bowlders and gravel is now being transformed into extensive stretches of green meadow, yielding plentiful crops, and at the same time confining the river within a permanent and defined bed in a way no series of expensive embankments could easily have affected. The result of river embankments has been too often to raise the bed year by year, so that they too require to be raised. In the Moselle valley, on the other hand, the floods are allowed to flow almost unchecked over the whole of their old channel; but when they retire they leave beneficial results instead of injury behind them, and resume the same course as they did before they rose.

This work was commenced by two brothers, Messrs. Dutac, in 1827, by their buying fifty acres along the left bank of the river's bed at LaGosse, a little below Epinal. At the head of this a rough bowlder dam was thrown across the river, turning about 70 cubic feet per second of its waters into a channel taken along the left of the estate. To this was given a gentle slope, which soon raised it above the river; and when lately seen the whole of the land lying between the river and the canal was a fine green meadow. The masonry works on the canal are all of the simplest kind, and require no remark save to notice this simplicity. The process then is as follows:—Below the dam there is erected an embankment at such points as are required, high enough to prevent the full current of the river from anywhere sweeping over the land to be reclaimed, but not at all intended to keep it from being flooded. From the main canal are taken out little branches, and the land to be irri-

gated by them is carefully leveled in a succession of parallel ridges and valleys running at an angle to these branches. About every 25 feet along their course are little openings, admitting a stream of water about six inches wide and half as deep, which flows along and overflows a channel made on each ridge, running over the slopes into a similar channel in the depression below. Along this it runs into a catch-water drain, which collects all these little separate streams, and a little farther down commences to give the water out again to irrigate a fresh piece. Sometimes the irrigating streams are made in pairs, back to back, sometimes they run singly.

FIG·12



The annexed diagram, Figure 12, is taken from a sketch made on the spot; *a* is the main canal, *b* the distribution channels, from which the water flows into the minor channels *c*, and over the ground on each side down into the dips, where the minor drainage lines *d*, carry it off to the main drain *e*, which, at a lower level, becomes in turn a distributing channel, repeating the operation. The main line *a*, diminishes at last into a distribution channel *b*, and that in time into minor channels. Of course it requires a good deal of labor to bring the gravelly bed into shape for this method of watering, but once done there is very little further outlay.

It is then sown with grass seed (without making any attempt to clear it of stones), and the irrigation is at once commenced. A light deposit of mud forms, every flood increases it, the irrigation is carried on incessantly, and the grass soon begins to sprout.

The silt deposit proceeds fast at first where the water proceeds directly through the gravel, which acts as a filter. By degrees this filtration causes a nearly impermeable bed, through which very little of the water escapes, and just so much the more flows by the drainage-lines, and flows off without having entirely divested itself of its particles of mud. Were it not for this the meadows would rise higher each year and soon be above the water's reach, but it is found that after a few years there is no sensible change in their level, and what fresh silt is deposited only makes good what is consumed on the vegetation.

There are in America large areas of alkali land within the irrigation districts, which would be benefited by this method of silting up.

Article 22. Equalizing Cuttings and Embankments.

The cross-section of the water channel and its slope, or grade being determined, the next step is to fix the depth of digging.

The cross-section of the canal can be fixed so that the surface of the water may be:—

1. *Within soil*, or, in other words, all in cutting.

2. *Above soil*, so that all the water is carried by embankments.

3. Partly in cutting and partly in embankment, or in cut and fill.

In some cases, for sanitary reasons, or in very pervious soil, not suited to make good banks, where the

loss of water and the cost of repairs to banks are serious, it may be necessary to keep within soil. Care must be taken, however, by sinking trial pits, that a sandy stratum is not reached by deep cutting, as, in this event, much water may be wasted by absorption, and the formation of swamps may seriously affect the health of the district, and ruin land, by water-logging, for any useful purpose.

In the second case, where the canal is all in embankment, there is always danger of breaches and consequent damage, and also the stoppage of irrigation when urgently required. In some soils the banks may require to be puddled.

The third case has several advantages. When the canal is partly in cut and partly in fill, the water has usually sufficient elevation above the land to give a command of level for purposes of irrigation.

It is also the most economical channel, as the cross-section can be arranged so that the earth excavated from the channel suffices for the banks, due allowance being made for shrinkage and waste.

It has a further advantage, where saving of time is an object in completing a work, as there is less material to be moved than when the canal is all in cut or all in fill.

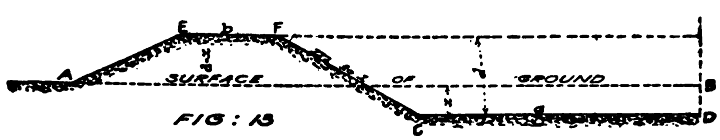


FIG: 13

The diagram, Figure 13, shows a cross-section of half of a canal, not drawn to scale, where the excavation is sufficient to make the banks, due allowance being made for shrinkage.

AB shows the surface of ground which is assumed to be level.

x = depth of digging which is required.
d = depth from top of bank to bed of canal.
$(d - x)$ = depth from top of bank to surface of ground.
a = CD = half bed-width of canal.
m = ratio of slopes CF = AE, that is, the ratio of horizontal to vertical distance of slope as, for instance, 2 horizontal to 1 vertical, then $m = 2$.
b = EF = top width of bank.

As the area of the excavation is to be equal to that of the embankments, we have:—

$$(x \times a) + \frac{x \times x\,m}{2} = b \times (d - x) + (d - x) \times (d - x)\,m, \text{ that is,}$$

$$x^2 m - x\,(2a + 2b + 4\,d\,m) = -\,2\,b\,d - 2\,d^2 m.$$

Now, let $a = 40$ feet, $b = 6$ feet, $d = 7$ feet, and side slopes 2 to 1, that is, $m = 2$, and substituting these values and reducing, and we have:—

$$x^2 - 74\,x = 140.$$

$$\therefore\ x = 1.943 \text{ feet.}$$

Figure 14 shows a cross-section where a berm at ME is required.

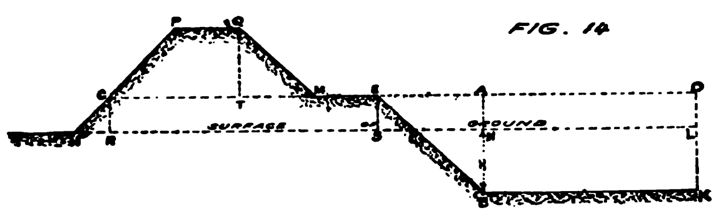


FIG. 14

The surface of the ground is assumed to be horizontal at HL.

Let a = BK = half width of canal bed.

d = AB = depth of canal from surface of berm to bed of canal.

$x = \text{NB} =$ required depth of digging to give sufficient material to make the bank.

$\text{A} =$ area of bank above EC.

$\text{B} =$ area of canal below DE.

Then, whatever the position of the natural surface, A and B are constants.

It is required to determine the depth, BN, or x, so that the area of excavation BFLKB, shall be equal to the area of embankment EFHPQME, that is:—

$$\text{B} - \text{EFLDE} = \text{A} + \text{EFHCE}, \text{ that is:—}$$

$$\text{B} - \frac{\text{ED} + \text{FL}}{2} \times (d-x) = \text{A} + \frac{\text{EC} + \text{HF}}{2} \times (d-x), \text{that is:}$$

$$\frac{d-x}{2} \times \left\{ (\text{ED} + \text{FL}) + (\text{EC} + \text{HF}) \right\} = \text{B} - \text{A}$$

Now let $\text{EC} = w$, and

$\beta =$ angle of BE and MQ with horizon, and

$\theta =$ angle of HP with horizon.

Then $\text{HR} = \text{CR}. \cot \theta = (d-x) \cot \theta$

$\text{SF} = (d - x) \cot \beta$, and

$\text{HF} = w + (d - x) \times (\cot \beta + \cot \theta)$

$\therefore\ \text{EC} + \text{HF} = 2w + (d - x) \times (\cot \beta + \cot \theta)$

Now $\text{ED} = \text{AD} + \text{AE} = a + d \cot \beta$

and $\text{FL} = \text{NL} + \text{NF} = a + x \cot \beta$

$\therefore\ \ \text{ED} + \text{FL} = 2a + (d + x) \cot \beta$

Substituting the values of (EC + FH) and (ED + FL) in equation, and we have:—

$$\frac{d-x}{2} \times \left\{ 2a + (d+x) \cot \beta + 2w + (d-x)(\cot \beta + \cot \theta) \right. = B - A$$

$$\therefore \frac{x}{2} \cot \theta - x(a + d \cot \beta + w + d \cot \theta)$$

$$= B - A - d(a+w) - d^2 \cot \beta - \frac{d^2}{2} \cot \theta$$

From this equation the value of x can be found.

Example:—Given $\beta = \theta = 45°$ and $\cot \beta = \cot \theta = 1$

Let $a = 50$ feet, $d = 8$ feet, $w = 40$ feet, PQ = 25 feet, QT = TM = 6 feet $\therefore$ CM = 37 feet.

$$\text{Then } B = ad + \frac{d^2}{2} = 432$$

$$A = \frac{25+37}{2} \times 6 = 186 \therefore \text{ equation becomes}$$

$$\frac{x^2}{2} - 106x = 432 - 186 - 720 - 64 - 32$$

$$\text{and } x = 5.53 \text{ feet}$$

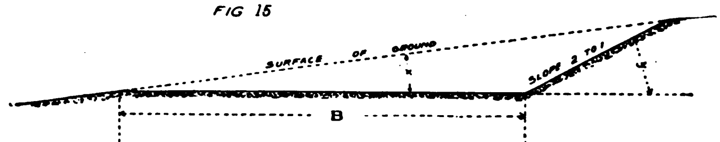
Having determined the depth x, in either case, then an addition to that depth has to be made, in order to compensate for the *shrinkage* of the material and the waste.

Article 23. Canal on Sidelong Ground.

It sometimes happens that the headworks of a canal are so located that the canal before it reaches the plains has to follow along steep side-hill ground.

As a rule, in such sloping ground, it will be more economical to have a deep narrow channel than the usual wide and shallow channel suitable for the plains. A section with bed width equal to or about twice the depth is better adapted to steep ground than one with a greater bed width to depth.

FIG 15



The cross-section of a canal in sidelong ground is either all in cut, or partly in cut and partly in fill. In each case the upper part of the cut is triangular in shape with a horizontal base as shown in Figure 15. The outer side of the triangle has the slope of the natural ground, and the inner side the slope of the inner side of the canal in cut.

Table 14, given herewith, will facilitate the computation of the triangular portion.

Let B = width of base.

A = area of triangle.

x = angle of natural ground with horizon.

y = angle of side slope of cutting with horizon.

k = co-efficient, for value of which see table.

$$A = \frac{1}{2}\left(\frac{B}{\cot x - \cot y}\right)^2 = B^2 K$$

TABLE 14. Giving Values of the Co-efficient K.

Angle of ground x in degrees	Values of K for different slopes. ¼ to 1	½ to 1	1 to 1	1½ to 1
1	.00877	.00880	.00888	.00896
2	.01761	.01777	.01809	.01842
3	.02655	.02691	.02765	.02844
4	.03558	.03623	.03759	.03906
5	.04472	.04575	.04793	.05035
6	.05397	.05646	.05873	.06239
7	.06334	.06541	.06999	.07525
8	.07283	.07558	.08176	.08904
9	.08246	.08600	.09410	.10387
10	.09220	.09670	.10700	.11990
11	.10215	.10765	.12064	.13720
12	.11230	.11900	.13510	.15620
13	.12250	.13050	.15008	.17661
14	.13290	.14240	.16610	.19920
15	.14359	.15470	.18300	.22404
16	.15430	.16720	.20080	.25120
17	.16551	.18045	.22018	.28241
18	.17660	.19370	.24070	.31640
19	.18838	.20797	.26257	.35617
20	.20000	.22220	.28570	.40090
21	.21230	.23753	.31151	.45261
22	.22520	.25380	.33890	.51280
23	.23743	.26942	.35688	.58445
24	.25000	.28570	.40120	.67020
25	.26391	.30405	.43687	.77624
26	.27770	.32250	.47610	.90900
27	.29194	.34186	.51942	1.08171
28	.30670	.36200	.56750	1.31230
29	.32173	.38340	.62185	1.64652
30	.33730	.40580	.68300	2.15510
32	.37030	.4545	.8333	
34	.4058	.5091	1.0373	
36	.4440	.5707	1.3297	
38	.4854	.641	1.7857	
40	.5307	.7225	2.6041	
42	.5807	.8183		
44	.6364	.9345		
46	.6983	1.0729		
48	.7692	1.25		
50	.8488	1.475		

Article 24. Shrinkage of Earthwork.

In the construction of embankments with earthy matter, sandy loam and similar materials, whether for canals or reservoirs, due allowance should be made for the shrinkage or settlement of the material.

The following extract, on this subject, is from a paper by the writer, on the *Shrinkage of Earthwork*, published in the Transactions of the Technical Society of the Pacific Coast of June, 1885:—

"Books of reference in the English language usually give the shrinkage of different materials, without making any allowance on account of different methods of construction and different heights of bank. For instance, the shrinkage of earth in general is given at about 10 per cent. Now, if 10 per cent. be sufficient for the shrinkage of a bank of that material, and 30 feet in height, constructed from the end of bank to the full height by "tipping" from wagons, surely a similar bank only 12 feet high, built up in layers, and consolidated by good scraper work, will shrink much less than 10 per cent.

"In no other branch of Civil Engineering, since the time when railroads were first commenced, has such an immense quantity of work been carried out, and expenditures incurred, as in earthworks; and in no other branch of engineering, of equal importance, have so few experiments, on a scale adequate to the interests involved, been published. In other branches of engineering, long, tedious and expensive experiments are carried out without any other return resulting from them than the information they give; but experiments on earthwork could be carried out on a large scale, as actual work, and with little, if any, additional expense more than the contract price of the work.

"Some of the materials are mentioned more than once, in the table given below, with a slight change in name, but the writer deems it better to give each author's own words descriptive of the material than to make a selection of the materials under a fewer number of names.

TABLE 15. Giving Shrinkage of Different Materials.

MATERIAL.	AUTHORITY.	Percnt'ge of Increase + or Diminution — of Embnkme't to excv'tion	REMARKS.
Sand	Hewson	— 10	
Very light sand	Graeff	— 10	
Light sandy earth	Morris	— 12.5	
Light sandy soil	Molesworth	— 11	
Gravel and sand	Vose	— 9	
Sand and gravel	Trautwine—Searle	— 8	
Earth	Miss. Levees, 1882		1-5 addition to height of bank
Earth	Simms	— 10	
Earth (scraper work)	Canadian Pacific R. R.		Shrinkage of bank 10 %.
Earth (grading machine)	Canadian Pacific R. R.		Shrinkage of bank 15 to 17 %.
Earth (carefully tamped)	Graeff	— 9 to — 20	
Loam & light sandy earth	Vose	— 12	
Loam	Trautwine—Searle	— 12	
Clay and earth	Vose	— 10	
Yellow clayey earth	Morris	— 10	
Gravelly earth	Morris	— 8.5	
Gravel	Molesworth—Vose	— 8	
Clay	Hewson		1-6 addition to height of bank
Clay	Trautwine—Searle	— 10	
Clay before subsidence	Molesworth	+ 20	
Clay after subsidence	Molesworth	— 8	
Puddled clay	Trautwine	— 25	
Wet soil	Searle	— 15	
Loose vegetable surf. soil	Trautwine	— 15	
Chalk	Molesworth	+ 30	
Rock	Vose	+ 50	
Rock	Graeff	+ 50 to + 60	
Rock	Rhine Nahe Railroad	+ 25	
Rock	Trautwine	+ 66 to + 75	
Rock, large fragments	Searle	+ 60	
Hard sandstone rock, large fragments	Morris	+ 42	
Blue slate rock, small fragments.	Morris	+ 60	
Rock, large blocks	Molesworth	+ 50	
Rock, medium fragments	Searle	+ 70	
Rock, medium unselected	Molesworth	+ 25 to + 30	
Rock (metal)	Molesworth	+ 20	
Rock, small fragments	Searle	+ 80	
Rock fragments (loose heap)	Trautwine	+ 90	
Rock fragments (carelessly piled)	Trautwine	+ 75	
Rock fragments (carefully piled)	Trautwine	+ 60	
Rock with considerable clay	Graeff	+ 0	

Article 25. Works of Irrigation Canals.

The works of irrigation canals include, weirs, dams, regulators, sluice-gates, scouring-sluices, movable dams, bridges, culverts, aqueducts, superpassages, flumes, inverted syphons, level crossings, inlets, drops or falls, rapids, tunnels, escapes or wastes, silt-traps or sand boxes, retaining walls, modules for measuring water, cuttings, embankments, and, on navigable canals, locks. It is very seldom, however, that a canal has all the above works. These works are described, somewhat in detail, in the following pages.

Article 26. Wells and Blocks.

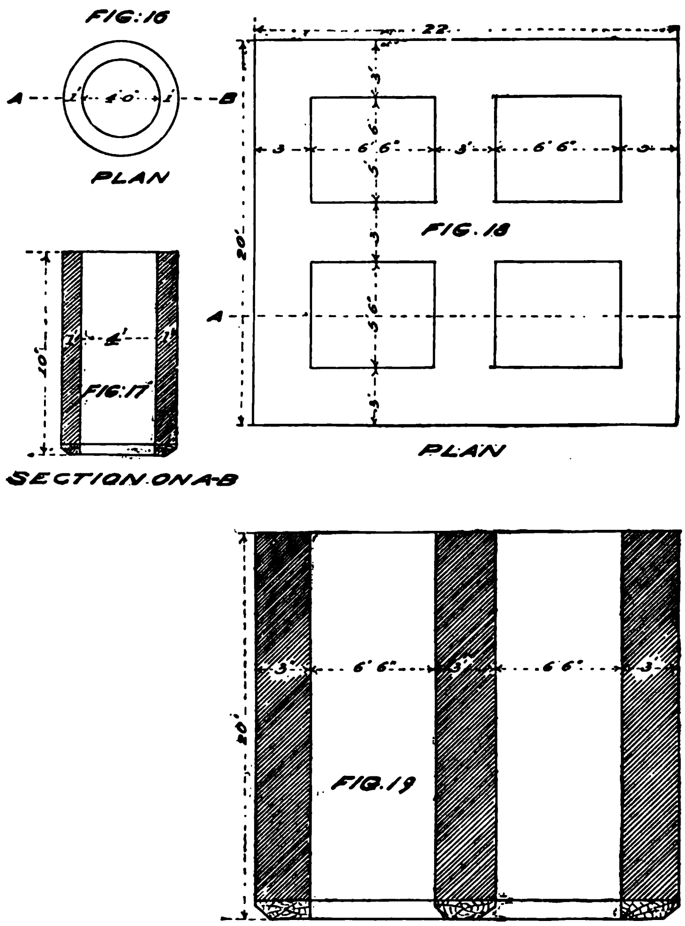
As wells and blocks are frequently referred to in the descriptions of the foundations of works in India, a brief description of them is herewith given.

Wells for foundations are usually brick cylinders, which are sunk to a certain depth in a sandy river. After they are sunk to the required depth they are filled, or partly filled, with concrete. When the lower part only is filled with concrete, the upper part is filled in with sand over the concrete. In addition to being a foundation for weirs, wells also diminish the cross-sectional area of the bed of the river through which percolation takes place.

A *block*, as its name implies, is a block of masonry having one or more vertical holes through it. Blocks answer the same purpose in every respect as wells. Figures 16 and 17 show a plan and cross-section of one of the wells under the walls of the Sone Weir, shown in Figure 37, and Figures 18 and 19 show a plan and cross-section of one of the blocks under the piers of the Solani Aqueduct, shown in Article 34.

The method pursued in sinking them is as follows:—

WELLS AND BLOCKS



SECTION ON A-B

If the wells to b. constructed and sunk are on a sandbank in the river bed, which is dry, the sand is excavated until water is reached, then the well-curbs are placed on the level of the water, and the masonry of the well is commenced; but if a stream has to be crossed, it is

diverted from that part of the river, after which the water is dammed and stilled. This is, of course, often a very difficult operation where the bed of a river consists of sand to a depth of, perhaps sixty feet. After the water is stilled, sand is thrown in, and an embankment formed across it, sufficiently wide to found the wells on; they are then built on it and afterwards sunk.

The wells are allowed to stand from ten to fifteen days after being built, to allow the masonry to set. The wells are then sunk by excavating the sand from within them, it being generally found that the quantity of excavation is about double the cubic area of the well sunk. The wells under the walls of the Sone Weir, Figures 16 and 17, are six feet wide on exterior diameter, and are sunk from eight to twelve feet below low water mark. These wells are sunk in single rows, each well being separated from the next one, in the line crossing the river, by a space of about six inches. The inside of the wells and the craters all around them are then filled in with rubble stone, the surface to a depth of two feet inside, and between, the wells being filled with concrete. Large stone slabs are then placed over the top of the wells, binding the walls to the hearting, and also bonding them to one another, and the masonry of the well is then commenced.

Wells have been sunk for foundations of bridges in sandy rivers, to a depth of over seventy feet.

Article 27. Headworks of Irrigation Canals.

The works at the head of a canal, for regulating and controlling the quantity of water required to be admitted to it, consist of a Weir across the river, by which the water is checked and diverted into it, and a Regulator across the head of the canal, by which the proper quantity of water is admitted.

In the Regulator are fixed sliding gates, or some other device, to control the supply of water to the canal, and in the weir and near the head gate is placed a Scouring Sluice to control somewhat the flow of water in the river past the head gate.

The operation of the Weir, Scouring Sluices and Regulator is so intimately connected, that a description of one of them applies more or less to the others; therefore, the descriptions given below, in the articles entitled *Diversion Weirs, Scouring Sluices and Regulators*, are only descriptions of different parts of the Headworks.

The requirement for good headworks for an irrigation canal are the following—but these are seldom to be found in one place:—

1. Permanent banks, and bed, which will prevent the river from eroding the banks and endangering the regulator, etc.

2. A straight reach of the river for say half a mile up and down the river from the weir.

3. A velocity in the river as low as, or not much greater than, the velocity in the canal. The nearer the velocity in the canal approaches to that of the river, the less silt will be deposited in the former.

4. That the current of the river should flow at right angles to the center line of the canal at its head.

5. That the river at the headworks, and after the construction of the weir, shall not overflow its banks.

6. That the bank of the river at the regulator is not very high, so as not to involve very heavy digging for the first few miles of the canal.

With reference to the third requirement mentioned above, Mr. C. E. Fahey, M. Inst. C. E., states:*

*Transactions of the Institution of Civil Engineers, Vol. 71.

"If the velocity (in the river) across the mouth of a canal exceeds the proposed velocity in the canal, the result must be that the latter will soon silt up. Of course some silt will deposit in all but the largest canals, in which a high velocity can be kept up; but if a canal is led off from a point in the river where the velocity is from five to six feet per second, the water (in the Indus) at this point will have its full proportion of silt in suspension, and the heaviest part of this silt, namely the sand, which the above velocity was able to keep in suspension, will drop in the mouth of the canal, where the velocity is suddenly reduced to about three feet per second. This fact admits of no dispute. It is proved every year in the Sind Canals. If a canal is in fair order, that is, if it has a properly regulated width and bed-slope, the sandy deposit will be distributed along the upper third of the canal, the heavier sand in the first mile or so, the finer lower down, and the clay at the extreme tail, while the central portion will seldom or never require cleaning. Although the velocity in the canal is not sufficient to carry on the sand, it is sufficient to carry on the clay, and if only escapes could be provided at the tails of all canals, which is not practicable in Sind, there would be no clay to be annually removed."

Article 28. Diversion Weirs.

WEIRS—DAMS—ANICUTS—BARRAGES.

A *Diversion Weir* is a weir built across a river to *divert* the water into the canal. At certain times, and always during floods, the water flows over part or the whole of this weir.

A *Reservoir Dam* is used to impound water, and, except in very rare cases, no water flows over its top. In engineering literature the terms weir, dam, anicut in

6

Madras, and barrage in Egypt, are also used to designate a weir across a river.

The cross-sections of diversion weirs are as different in form as the materials of which they are constructed. The drawings in this article give several examples, showing the sections adopted in different countries, to suit the material available for their construction, and their foundation in the beds of the rivers across which they are constructed.

Canals have frequently been taken off from rivers without weirs, but where these rivers are liable to change their beds by erosion of their banks, or where they carry large quantities of silt in suspension, it has been found impossible to regulate both the river channel and also the supply of water into canals on their banks, without a weir built right across the stream.

Some canals, without weirs, have their beds at the off-take, much lower than the beds of the river from which they derive their supply, with a view of obtaining a supply at the low stage of the river, but this is objectionable for several reasons, one is the great quantity of sand and silt likely to be carried into the canal and, therefore, the difficulty and expense of keeping the deep channel open.

In some cases, in Northern India, the canal is taken out of a branch of the main river; and the permanent diversion weir is thrown across the branch only, the water being diverted from the main stream into the branch by temporary dams constructed of bowlders, which are swept away on the rise of the river, and annually replaced. This arrangement has chiefly been due to the very heavy expense which would be incurred in throwing a permanent dam across the main river itself. An example of this method was in operation a few years since at the headworks of the Upper-Ganges Canal shown in Figure 26.

Dams in rivers are made solid, except at the scouring sluices, when they are called weirs. Of these, Figures 37, 39 and 43 are good examples.

When they are provided with openings through their whole length, or the greater part of their length, they are called dams in India. Indeed the term dam is always, in Northern India, understood to mean an *open* dam, or one partly open and partly closed. Examples of this latter class of dam are to be found in the Kern River dam, Figure 20, the Myapore dam, Figure 27, and the Barrage of the Nile, Figure 32.

The advantage of the *Weir* is that it is *self-acting*, requiring no establishment to work it, and if properly made ought to cost little for repairs. It is also a stronger construction, better able to withstand shocks from floating timbers, etc. Its disadvantages are, that it causes a great accumulation of silt, bowlders, etc., above it, and interferes far more than an open dam, with the normal regimen of the river. It is possible, that in certain cases, this might result in forcing the whole or part of the river water to seek another channel, and the possibility of this should always be taken into account; but if the river has no other channel down which it could force its way, the accumulation of material above the weir would be an advantage rather than otherwise, as adding to its strength.

The advantage claimed for the open dam is that the interference with the normal action of the river is reduced to a minimum, the strong scour obtained by opening its gates effectually preventing any accumulation of silt above.

A *dam*, in India, consists of a series of piers at regular intervals apart, on a masonry flooring carried right across and flush with the river bed, protected from erosive action by curtain walls ol masonry up and down stream.

The piers are grooved for the reception of sleepers or stout planks, by lowering or raising which the water passing down the river is kept under control. The intervals between the piers may be six to ten feet, which is a manageable length for the sleepers. If the river is navigable at the head, one or two twenty feet openings fitted with gates must be provided to enable boats to pass.

The flooring must be carried well into the banks of the river on both sides, to prevent the ends of the dam being turned, and the banks and bed of the river will generally require to be artificially protected for some distance, above and below the dam, to stand the violent action of the water when the gates are partially closed.

The two flanks of the dam for some length are generally built as weirs; that is, instead of having piers and gates, the masonry is carried up solid to a certain height so that when the water rises above that height, it may flow over the top of it. The advantage of this arrangement is, that it affords an escape for water in case of a sudden flood when the dam may be closed, while, when the water is low, they keep it in the center of the river and away from the flanks, and thereby create a more perfect scour.

When the river is subject to sudden and violent floods, damage might be done before the sleepers could be all raised, one by one; it is better therefore to employ flood or *drop-gates* in such a case; that is, gates which turn upon hinges in the piers at the level of the flooring and which when shut are held up by chains against the force of the water. In case of flood, the chains are loosened, the gates drop down, and the water flows over them. Should the intervals between the piers be over ten feet, there would be a difficulty in hauling the gates up again.

A bridge of communication may be made between the

piers of the dam if required. But as it is not desirable to have it obstructed with traffic, it may be merely a light foot-bridge, or the intervals may be spanned temporarily with spare sleepers.

The dam and regulator are generally close together and connected by a line of revetment wall, as shown in Figures 28, 31, 40 and 48.*

In some cases iron or stone posts were fixed on the crest of the weir. Planks laid horizontally are fixed in grooves in these posts to raise the water about two feet higher than the crest of the weir. These planks are removed before the occurrence of floods.

The greater number of the weirs or dams across Indian rivers, and almost all those of modern date, are located at right angles to the general direction of the rivers. It is well known that the tendency of *oblique* weirs is to divert the strongest stream, and consequently the deepest channel towards the bank on which the upper end of the oblique weir is situated. It was, no doubt, quite true that, in rivers where a good foundation could be obtained, there would be very little objection to oblique weirs; but in rivers such as those which had to be dealt with in India, with sandy beds and difficult foundations, they were very objectionable, for three reasons:—

Firstly, they induced currents parallel to the weir;

Secondly, they caused a deepening of the channel above the weir, near the up-stream end, which was dangerous; and

Thirdly, they raised the level of the water in the river at the lower end of the weir.†

Another reason is that they cost more than the straight

*Roorkee Treatise on Civil Engineering.

†R. B. Buckley, C. E., in Proceedings of I. C. E., Volume 60.

weir, and, therefore, for all these reasons the latter weir is preferred in India.

The location of the dam should be studied with a view to the avoidance of flooding the country above the dam in the high stages of the river. To prevent flooding, long and heavy embankments had to be made above the Narora weir.

In order to reduce the first cost of construction, it has become a custom to build bridges and dams across streams at the narrowest point available, or to contract the stream for that purpose. This frequently involves great difficulties to the engineer in laying the piers and abutments, and also brings in an element of danger by adding to the scouring effect of the waters in the contracted channel. Moreover, it generally produces evil effects by the formation of shoals below the scoured-out channel.

The proper location for such works, and especially for dams across a river with unstable banks, where the highest factor of safety is desired, is in the broad reaches of the stream, where the depth of water is usually less, and especially in places where a "bar" has already been formed across the river by natural causes.

The dam across a river is not only analogous to a "bar" formed by natural causes, but in the scheme of irrigation by gravitation it is a "bar," and should be located and treated as such. If this is done at a broad passage of the stream, or where it has its average width, the first cost of material and workmanship may possibly be increased beyond a similar work at a contracted passage; but this is not an absolute necessity, as many of the ordinary difficulties to be overcome by the engineer are much lessened, and danger to the work in progress, and when finished is much reduced during floods and ice-gorges. The adjacent banks are less liable to be torn

away, wing-dams are avoided, the levees are less expensive and less liable to abrasion and to crevasses, there is less cost for protecting works, and less cost of subsequent supervision and repairs.*

With three exceptions, none of the weirs described in this article raise the water higher than fourteen feet. Indeed, all the weirs in the wide, sandy rivers of India are low weirs of the type of the Okhla Weir, Figure 39, in Northern India, and of the Godavery Anicut, Figure 44, in Madras.

The high weirs, the Turlock Weir, Figure 45, and the Henares Weir, Figure 46, have a cross-section not in favor in India. It is very likely that an Indian engineer would reverse these cross-sections and place the vertical side down-stream, with a water-cushion on the lower side, to receive the falling water and diminish its destructive effect. This will be referred to when describing the dams mentioned.

The Cavour Canal weir has a cross-section similar to the Ogee falls first constructed on the canals in Northern India. These falls destroyed themselves, and they had to be replaced by vertical falls with water-cushions. This is referred to in the article entitled *Falls.* Two of the weirs have *vertical drops*, the Streeviguntum Anicut, Figure 41, and Narora Weir, Figure 43. The latter, however, has a water-cushion three feet in depth at the low stage of the river, while the apron of the former is laid at the level of the low-water of the river.

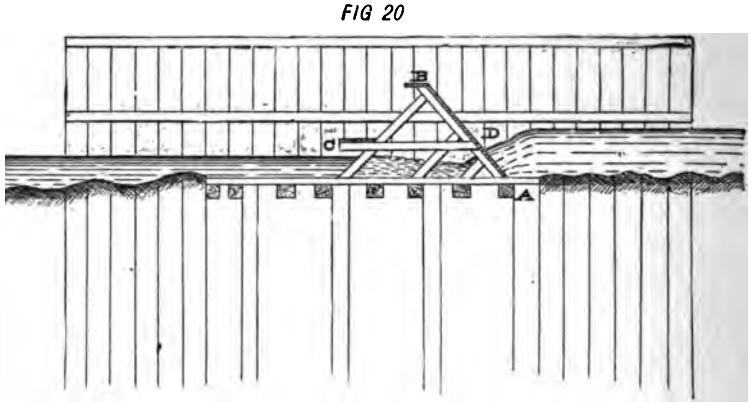
In America there are numerous dams of a temporary character which are made of brush and bowlders. At Phœnix, Arizona, dams are formed of stakes, brush and bowlders, rendered water-tight by filling in up stream, with gravel and sand. Stakes are first driven

*Irrigation in India, Egypt and India, by Professor George Davidson.

across the channel, and between these, bundles of fascines of willow trees, about three inches in diameter at their butts, are laid, with butts down stream, and weighted with a layer of bowlders; tule reeds in bundles are also used, mixed with willow and cottonwood tree. In alternate layers the dam is built up to the height of five feet. The willows sprout and the whole forms a mass of living brush and bowlders. When the current is too strong for a man to withstand while driving stakes, cribs are made and floated out and sunk, as was done with the fascine dam at Merced Canal head, in California.

The cross-section of the weir of the Calloway Canal, across the Kern River, California, is shown in Figure 20. The plan of the head works of this canal is shown in the article entitled *Methods of Irrigation.*

FIG 20



The Calloway Canal is diverted from the right bank of the Kern River, a few miles above Bakersfield. The average maximum discharge during the rainy season is probably over 19,000 cubic feet per second. The water of the canal is diverted from the river by a *very light*, open, wooden weir, extending at right angles to, and entirely across, the river from bank to bank. The length

of this diversion weir is 400 feet. The weir rests on three rows of 4"x12" anchor piles at right angles to the course of the river, and two rows of 4"x12" sheet piling, at the wings parallel to the course of the stream. The piles are driven ten feet into the bed of the river. On the bed of the river and resting on the tops of the piles, and on the mud sills, to which it is securely spiked, a flooring of plank two inches thick is fixed. This floor is about thirty feet in length in the direction of the river.

The trestles, *A*, *B*, *C*, *D*, Figure 20, are about four feet from center to center. These trestles support the planking, *A*, *B*, two inches thick, which holds up the water and thus diverts it into the canal. There are two light foot-bridges on the weir shown at *B*, and *C*. All the planking, *A*, *B*, are shown in position. When this happens the water on the up-stream side of *A*, *B*, is level, but as shown in Figure 20 some of the planking, not on the line of the cross-section are assumed to be out and water is flowing through the weir.

One man, standing on the foot-bridge, operates the two-inch flash-boards with an iron hooked rod. The total height of the weir is ten feet above the floor.

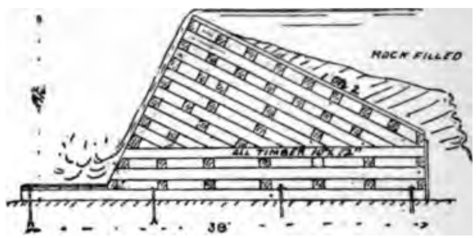
The Head-Sluice or Regulator, at the head of the Calloway Canal, is of similar construction to the weir just described, but exceeding it by one foot in height.

These head-works are in use seven years and they are reported to give satisfaction. As the whole weir is open in flood time the river bed above it has not silted up.

There is not, probably in the world, a lighter or cheaper weir, of an equal length, and situated on the sandy bed of a river, that has acted so efficiently, and that costs less, for its operation and maintenance. It was a bold undertaking to attempt to control such a river, having a flood discharge of over 19,000 cubic feet per second, with such a light structure, and it well exemplifies one of the *Peculiarities of American Engineering.*

The cross-section of the weir of the **Bear River Canal**, in Utah, is shown in Figure 21.

FIG. 21



Cross Section Bear River Weir.

Its location is well selected, as it has high rock abutments and a rock foundation. It is constructed of crib-work of sawn lumber. Between the crib-work it is filled in with earth and loose rock, and the up stream side, which has a slope of two to one, is filled in with rock. The down stream side has a slope of one-half to one. The lowest sills of the crib, ten inches by twelve inches, are drift bolted to the bed rock and on these planking is spiked, on the down stream side, to protect the foundation from the effect of the falling water.*

The weir across the North Poudre River, at the head of the North Poudre Irrigation Canal in Colorado is, in the center, thirty feet six inches high, and 150 feet wide on the top, and it is formed in two parts. The down-stream division, or face, which gives the necessary stability against floods, consists of crib-work and stones; the up-stream or back, which renders the weir water-tight, being a vertical panel or diaphragm of timber, backed with earth, small stones, gravel and mud, thrown in without puddling.

The crib-work is formed of round logs, ten inches at

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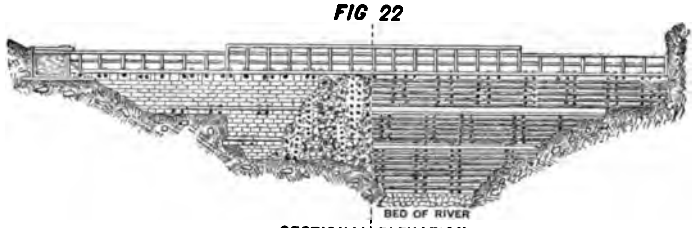
*American Irrigation Engineering by Mr. H. M. Wilson, M. Am. Soc. C. E., in Transactions of the American Society of Civil Engineers, Vol. 24.

CRIB DAM ON NORTH POUDRE IRRIGATION CANAL

FIG 22

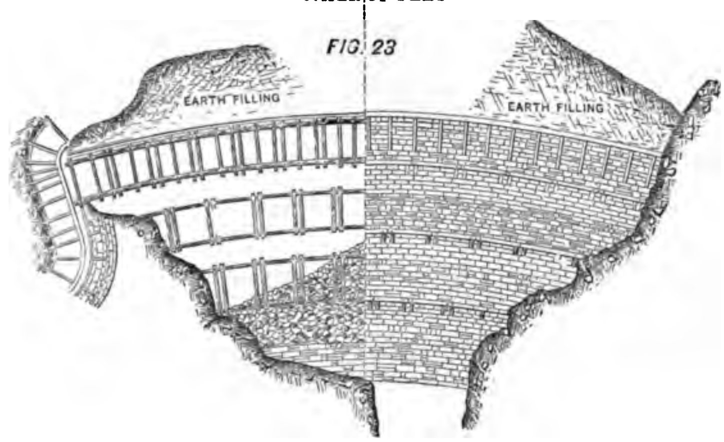


SECTIONAL ELEVATION

10 5 0 10 20 30 40 50 60 70 80

SCALE OF FEET

FIG 23



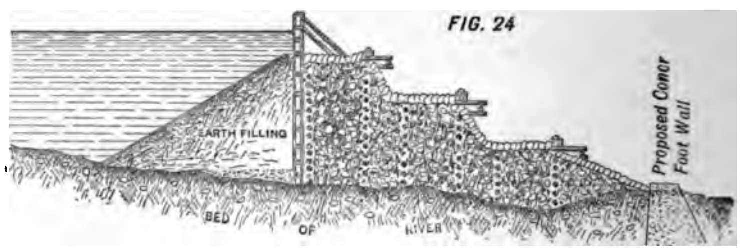
SECTIONAL PLAN

least in diameter, joined at the ends, as in ordinary log huts, with dovetail or tongue joints. Figures 22, 23, 24 and 25, give plans and sections of the weir. Each crib is ten feet long on the face, and is fastened together with eighteen-inch treenails, two inches in diameter. The cribs are radiated so as to form, when laid close together across the stream, curved tiers of 200 feet, 216 feet, and 232 feet radius on the face. There are three of these tiers, of different heights, six feet asunder. The interior of the cribs, and the spaces between the tiers, are filled with stones, and the exterior surfaces are faced with

large selected blocks of stone, carefully laid so as to overlap each other like the slates or tiles of a house, and without mortar. The arrises are protected by twelve-inch square blocks, securely bolted to the cribs. The timber diaphragm is carried four feet higher than the

CRIB DAM NORTH POUDRE IRRIGATION CANAL

Scale 32 Feet to one Inch

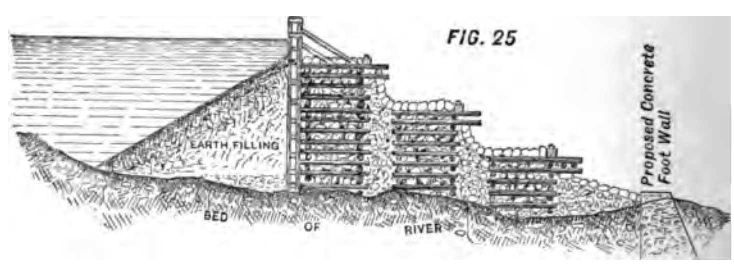


SECTION THROUGH CENTER OF CRIBS

cribs and stonework of the tallest tier, to form a "slash board," which can be removed in sections in case it is found liable to be damaged by ice. The center portion of the weir for a length of sixty feet, is carried two feet

CRIB DAM NORTH POUDRE IRRIGATION CANAL

Scale 32 Feet to one Inch.



SECTION AT ENDS OF CRIBS

higher than the sides, to throw the bulk of the stream on to natural benches of solid quartz rock on the sides, and thereby to protect the greater part of the face, and especially the toe in the center of the stream from the abrading power of the water.

The weir was founded on stone and debris, the depth of which had not been sounded, but it was hoped that

PLAN OF HEADWORKS OF UPPER GANGES CANAL.

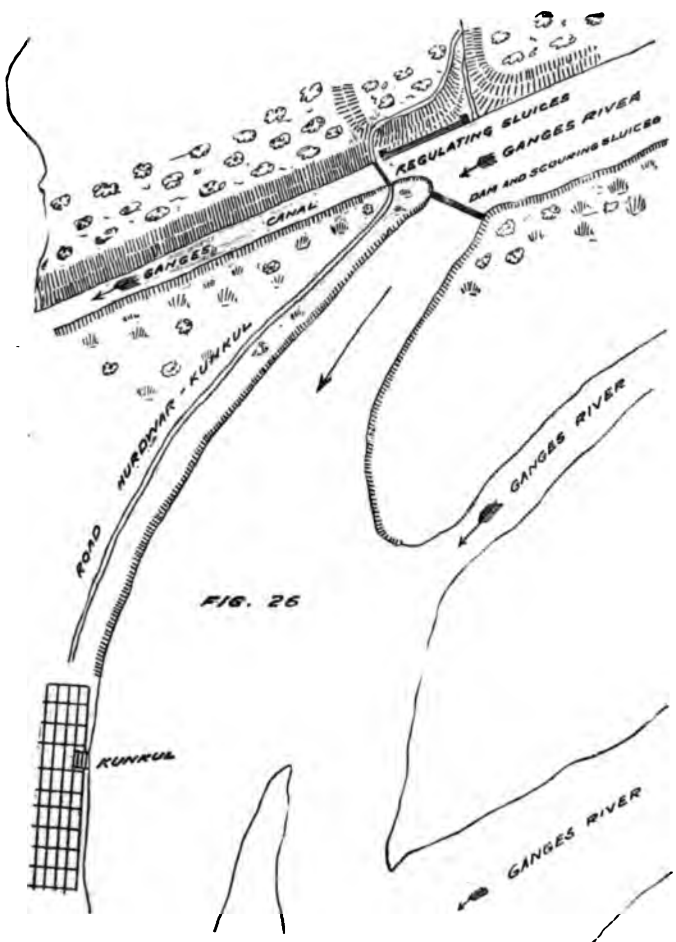
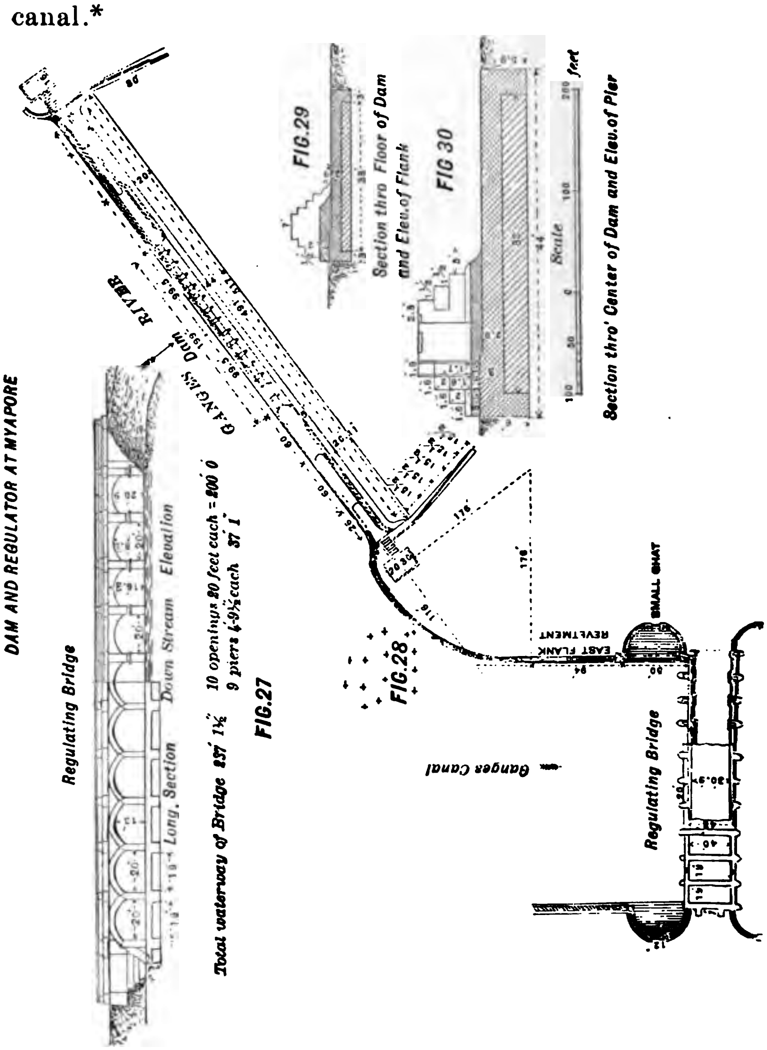


FIG. 26

the clay thrown into the back of the weir, combined with the silting up of the river, would have the effect of putting a stop to the flow of the water, and the result has justified the expectation. At first the water leaked

through and there was some difficulty in stopping it, but it was finally arrested. The weir was simply for the purpose of lifting the water high enough to enter the canal.*



*Irrigation in New Countries, by Mr. P. O'Meara, M. Inst., C. E., in Transactions of the Institution of Civil Engineers, Vol. 73.

The Myapore Headworks of the Upper or Original **Ganges Canal** are shown in the general plan, Figure 26, and in detail in Figures 27, 28, 29 and 30.

This weir is an example of an "open dam," differing from the unbroken "anicuts" of Madras, and the solid weirs with scouring sluices, such as the Sone, Okhla and Narora weirs, built in later years in Northern India. As stated by the designer, Sir P. Cautley, in the following extract, this dam is "in fact a line of sluices with gates or shutters, which are capable of being laid entirely open down to the bed of the river during the period of flood." This weir is designed somewhat on the plans of the Barrage of the Nile, a description of which is given below.

This dam differs also from the weirs now generally constructed in regard to its position in relation to the head sluices of the canal, which are, at Myapore, placed in a "*regulating bridge*," situated, not on the flank revetments immediately adjoining the weir abutment, but two hundred feet or more down the canal. This is a defective arrangement, as the *pocket* thus formed between the regulator and the actual commencement of the canal channel is filled by an almost still back-water, when the flood-waters are pouring over the dam; and this *pocket* becomes shingled up, with seven or eight feet in depth of bowlders and sand, marked * * * on Figure 28, and the supply, especially at low water, is reduced until this accumulation is cleared away.

The left flank of the dam abuts upon an island, in which nearly one-half of its full width is excavated.

The flooring of the dam and of the regulating bridge are laid on one level, and the front line of the latter is the zero to which the whole line of the canal is referable, as to levels and length. The zero point for levels

was fixed at the level of the bed of the river at the location of the regulator.

The dam itself, which is 517 feet between the flanks, is pierced in its center by fifteen openings of ten feet wide each; the sills or floorings of each opening being raised two and a-half feet from the zero line. These floorings are so constructed, that, if necessary, they may be removed, and a flush waterway be obtained as low as zero. The piers between the above openings are eight feet in height, so that the elevated flooring leaves the depth of sluice-gate equal to five and a-half feet. The piers are fitted with grooves for the admission of planks.

The Regulating Bridge, at the head of the canal, has ten bays or openings each twenty feet in width and sixteen feet in height, each bay being fitted with gates and the necessary apparatus for opening or closing them.

The narrowness of the platform, only forty-four feet, contrasts strangely, at this time, with the width of other weirs, as for example the Sone, the Okhla and the Lower Ganges, shown in Figures 37, 39 and 43, but it must be remembered that the bed of the Ganges at Myapore consists of large and small bowlders, forming a natural talus or apron below the weir; and that owing to the backwater of the other open channels of the river, which form a water-cushion below the weir, the bed of the Myapore channel below it has a tendency to rise instead of being scoured away. The above arrangement is given to illustrate what kind of headwork was adopted when the Original Ganges Canal was projected, but of late years a new weir, across the whole Ganges River, has been constructed, two or three miles above the Myapore dam, which somewhat modifies the above arrangements.*

*Roorkee Treatise on Civil Engineering

The Barrages of the Nile, at its bifurcation at the Rosetta and Damietta branches, are open weirs or dams, provided with openings along their entire length. Since

PLAN OF PART OF THE NILE DELTA
SHOWING LOCATION OF BARRAGES AND CANALS.

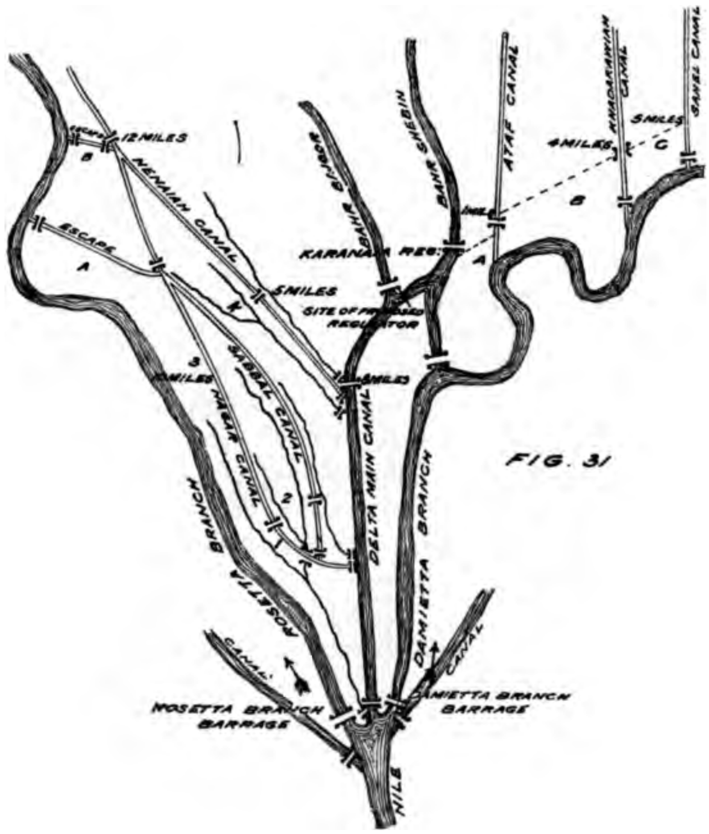


FIG. 31

the Nile in Egypt during flood, is considerably above the level of the country, which is protected by embankments from inundation, it would have been dangerous to build

a solid barrage, which would have still further raised the water surface, unless a length of barrage could have been obtained much in excess of the normal width of the river.

A plan of the head of the Delta of the Nile, showing the positions of the barrages, is given in Figure 31, and Figures 32, 33 and 34 give the plan and longitudinal and cross-sections of the barrage of the Rosetta Branch. A view of the barrage is given in Figure 35.

The Egyptians call the barrage the "Bridge of Blessings," for the reason that it has considerably extended the area of irrigation during the period when it is urgently required in Lower Egypt. The barrage crosses the Nile about twelve miles below Cairo, at the point where the river divides into two branches. The length of the western or Rosetta branch, following the sinuosities of its course, is about 116 miles, and of the eastern or Damietta branch, 124 miles. The plain which they traverse, termed the Delta, presents a front to the Mediterranean of about 180 miles, and forms by far the most valuable portion of the lands of Egypt.

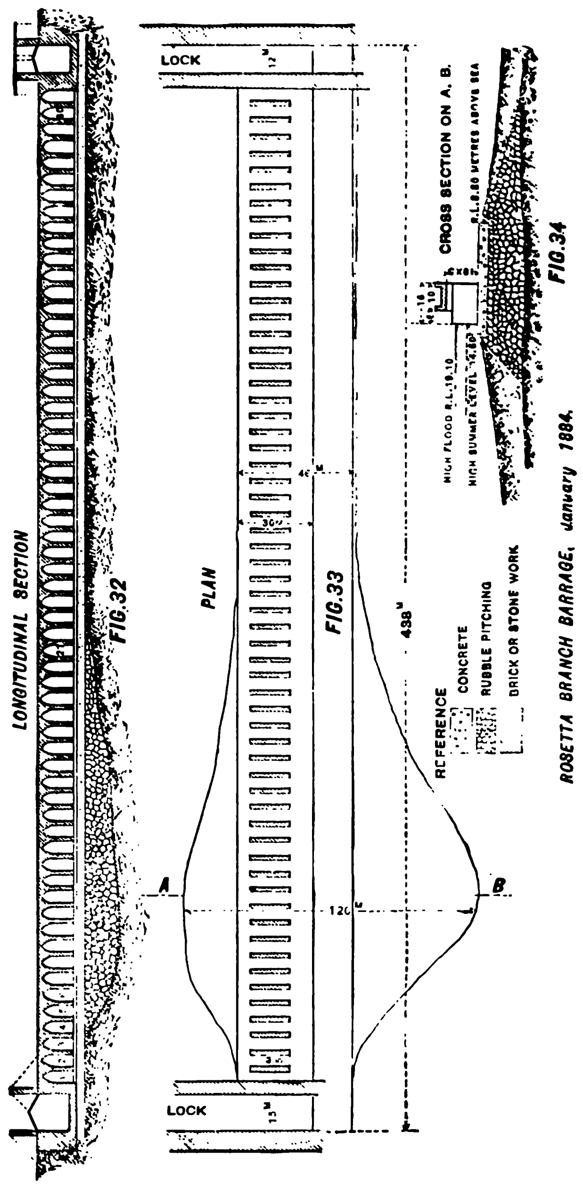
To form an idea of the barrage, with the aid of the drawings, imagine a bridge or viaduct of solid proportions established at the head of the delta, on each of the two branches of the river, and above these bridges the headworks of three great canals, destined to traverse in their course the Eastern, the Central or Delta, and the Western Provinces of Lower Egypt. If the arches of these bridges were closed by sluices, the water would of course be backed up and inundate the valley, unless it were carried off by the canals fed from the river and restrained by the banks formed to control its overflow. The water thus raised and thrown into the three canals, of which mention has been made, could then be discharged at will, on any of the lands of Lower Egypt

through openings made in the canal banks. When the Nile commenced to rise, the sluices in the arches of the bridge would be opened gradually, until at the time of the great floods, there would be no obstruction, except the piers of the bridges, to the passage of the waters. By this system it would be possible to regulate the height of water as desired, to increase the height of feeble floods, and to diminish somewhat the effect of violent floods by discharging water through the three main canals.

M. Mougel, an able French engineer, designed the barrages, and constructed them under great difficulties. During his absence from Egypt they were condemned as unsafe, and for twenty years, from 1862 to 1882, they were never used to raise the Nile water to anything like the height originally contemplated. About the latter date, General Sir C. C. Scott Moncrieff, R. E., took charge of the work, and since then he has so thoroughly repaired and strengthened the foundations of the barrages, that they now retain a head of water never attempted before he took charge of the works. After making a partial success of the barrages, General Moncrieff publicly acknowledged, in the most generous manner, the great ability of M. Mougel, the original designer of the works.

The following description explains the barrages as they existed, before the construction of the works to reinforce the foundations, carried out by General Moncrieff.

The Nile barrages are two open weirs thrown across the heads of the Rosetta and Damietta branches, at the apex of the Delta. Of the two branches the Rosetta has nearly twice the flood supply of the Damietta, while its bed is some six feet lower. The Damietta branch feeds eight important canals. The Rosetta barrage is 1,437 feet between the flanks, and the Damietta 1,709.


LONGITUDINAL SECTION
FIG. 32
LOCK
PLAN
FIG. 33
438 M
120 M
A
B
LOCK
CROSS SECTION ON A. B.
HIGH FLOOD R.L. 19.10
HIGH SUMMER LEVEL
R.L. 8.80 METRES ABOVE SEA
FIG. 34
REFERENCE
CONCRETE
RUBBLE PITCHING
BRICK OR STONE WORK
ROSETTA BRANCH BARRAGE, January 1884.

These barrages are separated by a revetment wall 3,280 feet in length, in the middle of which is situated the head of the Main Delta Canal. The platform of the Rosetta barrage is flush with the river bed, being 29.8 feet above mean sea level. Its width is 151 feet and depth 11.5 feet, and it is composed of concrete overlaid with brick and stonework as shown in Figure 34.

Down stream of the platform is an irregular talus of rubble pitching, varying in places from 150 to ten feet in width, and from fifty to two feet in depth. The left half of the platform is laid on loose sand, the right half on a barrier of rubble pitching overlying the sand. This loose stone barrier is thirty feet high, 200 feet wide at the deepest part, and tapers off to zero at the ends. It closes the deep channel of the river, and its only cementing material is the slime deposit of the Nile. This deposit is so excellent that the barrier is practically water-tight. The platform supports a regulating bridge with a lock at each end. This bridge consists of sixty-one openings each 16.4 feet wide. The lock on the left flank is 39.4 feet wide, while that on the right is 49.2 feet. Fifty-seven of the piers are 6.6 feet wide, while three of them are 11.6 feet wide; their height being 32.2 feet. The lock walls are 9.8 feet and 14.8 feet wide. The piers support arches carrying a roadway. The waterway of the barrage is 34,359 square feet, while the high flood discharge is 225,000 cubic feet per second, causing a banking up or afflux of 0.8 foot.

During the floods of 1867 the floor of ten openings of the Rosetta Barrage settled 0.4 foot, producing a deflection in the superstructure both horizontally and vertically, and after this time no attempt was again made to raise the water so high until after the completion of the remodeling of the foundations by General Moncrieff.

The Damietta Barrage has ten openings of 16.4 feet

each, more than the Rosetta Barrage. The platforms and superstructures are on the same level, and exactly similar.

FIG. 35. VIEW OF NILE BARRAGE.

The Okhla Weir, on the River Jumna, Figure 39, is a mass of loose rubble stone with absolutely no foundation, and holds up annually ten feet of water, when the water pressure per lineal foot bears to the weight of the dam a proportion of $\frac{3,125}{129,600}$ or $\frac{1}{40}$. Nile sand is much finer than that in the Jumna, and will therefore require a lower co-efficient.

Considering the barrage a thoroughly unsound work as to its foundations, and relying only on friction, it was determined to make the submerged weight of masonry bear a ratio of fifty to the pressure of the water going to be brought on it. Springs might cause a slight subsidence of any part of the barrage, but it could not be moved as a whole. The pressure of a head of ten feet of water would be 3,125 pounds per lineal foot. The

submerged weight of the platform, as first constructed, was 103,983 pounds per lineal foot. The co-efficient was $\frac{1}{33}$. That this proportion might be $\frac{1}{60}$, it was necessary to make the rubble talus everywhere 131 feet wide and ten feet deep, with a submerged weight per lineal foot of 51,668 pounds. This made the submerged platform and talus together 155,651 pounds as compared to the pressure, 3,125 pounds. Since only one-third of the talus was completed in 1884, the barrage was not required to hold up more that 7.2 feet of water, but on the completion of the talus in 1885, ten feet of water were held up.*

The headworks of the Sone Canals, taken from the river Sone, in India, shown in plan in Figure 36, is a good illustration of the headworks of a modern canal, taken from a river in the plains of India, and having scouring sluices with movable shutters.

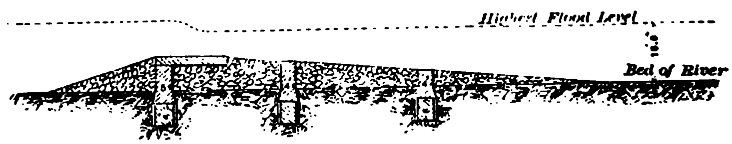
FIG. 36. PLAN OF HEADWORKS OF SONE CANALS.

The length of the weir between the abutments, on the right and left banks of the river, is 12,550 feet, or 2.35 miles, and its crest is eight feet higher than the bed of the river. As two canals are taken off above this weir, one from each bank of the river, there are two sets of end weir scouring sluices, one at each extremity of the

*Irrigation in Lower Egypt, by Mr. W. Willcocks, C. E., in Vol. 88 of Transactions of the Institntion of Civil Engineers.

weir. There is also a central set of scouring sluices to provide a greater control over the regimen of the river, and to assist in keeping open a navigable channel across it, between the locks of the two canals. However, after an experience of several years, they have been found insufficient for this purpose. The pool above the weir silted up so much that when the water was level with the crest of the latter, that is, when the water was eight feet above what used to be the bed of the river, it was with difficulty that a boat drawing three feet of water

FIG.37 SECTION

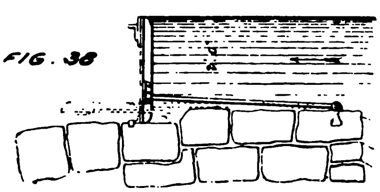


WEIR AT DEHREE SONE CANALS

could be got across from the canal on one side to that on the other. Many islands were formed one foot or two feet above the level of the crest of the weir, and were yearly increasing. To facilitate navigation, and to raise the level of the pool with the object of obtaining a greater depth of water upon the head sluices of the canals, it was decided to put a movable dam two feet high, along the whole length of the weir. Four men can raise these shutters, when a depth of six or eight inches of water is flowing over the crest of the weir almost as quickly as they walk.

MOVABLE DAM TO BE ERECTED ALONG THE CREST OF THE SONE WEIR.

FIG. 38



Each set of scouring sluices is made up of twenty-five movable shutters of a width of twenty feet each, that is, each set is 500 feet

in length. These movable shutters are explained in the article entitled *Sluices and Movable Dams.*

Before the construction of the weir the mean depth of the river at time of high flood was found to be 11.64 feet, and the breadth between the banks 12,400 feet.

The river in flood rises eight and one-half feet over the crest of the weir, and discharges about 750,000 cubic feet per second. Colonel Dickens estimated the flood discharge at 1,026,000 cubic feet per second, but his estimate was too large. The catchment basin of the Sone is about 23,000 square miles.

The weir is composed mainly of dry rubble, and is similar in cross-section to the Okhla weir, Figure 39, but differing from that structure in having foundations to its three parallel masonry walls, which traverse the mass of dry rubble from end to end, and keep this mass together.

An ample supply of good stone, both for rubble and ashlar, is obtainable from quarries about five miles distant. The Sone differs from the Himalayan rivers generally, in being confined within a permanent channel, so that no flank defenses of any importance are necessary on the banks of the river. The three parallel walls of the dam are founded on shallow, hollow blocks, sunk with the aid of Fouracre's excavators. These blocks have thin walls; for blocks of six feet interior width a single brick thick was sufficient, while for fourteen feet blocks the walls were built from one and one-half or two brick thick.*

In the Bengal Revenue Report of the Public Works Department for 1889–90, it is stated that:—

"For many years after the construction of the Sone Weir, the recurring failures of the piers of the river

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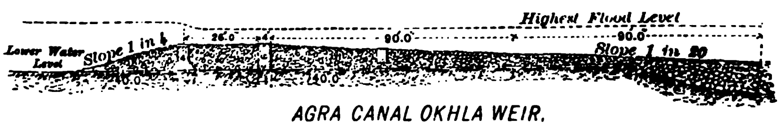
**Indian Weirs*, by Major A. M. Lang, R. E., Professional Papers on Indian Engineering, Vol. VI. Second Series.

sluices, owing to the inherent weakness of their design, were a constant cause of expenditure in repairs, and in 1885 it was decided to build them on a stronger model. The work is now completed."

The Okhla Weir, Agra Canal in India, is shown in cross-section in Figure 39. This is a remarkable work,

FIG. 39 SECTION



AGRA CANAL OKHLA WEIR.

in which the engineers of Northern India have exceeded the Madras engineers in the shallowness of foundation, in which the so-called "Madras system" was supposed to differ widely from the practice of other parts of India. In this case foundation may be said to be entirely dispensed with. The lowest cold water level, 649 feet above Kurrachee mean sea level, was adopted as the datum, and a trench was made for 2,438 feet across the dry sandy bed of the Jumna, eight miles below Delhi, at this level; and in this trench was built, in the winter of 1869–70, a wall four feet thick and five feet high of quartzite rubble masonry, laid in lime cement; a sloping apron of dry quartzite rubble extended five feet above this wall, and a sloping talus of similar material was laid for 100 feet below it; the floods of 1870 were allowed to pass over this weir, and left it unharmed. During the next winter, the wall was raised to its full height of nine feet, and the talus was lengthened to 180 feet. The floods of 1871 overtopped the weir by five and a-half feet, more than 1,000,000 cubic feet per second sweeping over it, while 40,000 cubic feet broke over the left shore enbankment and inundated a large tract of country. The greatest velocity was 18.6 feet

per second, and was found to be at forty-two feet below the crest. Stone was worked out of the talus, and deep holes, twenty feet deep, were scoured out on the down-

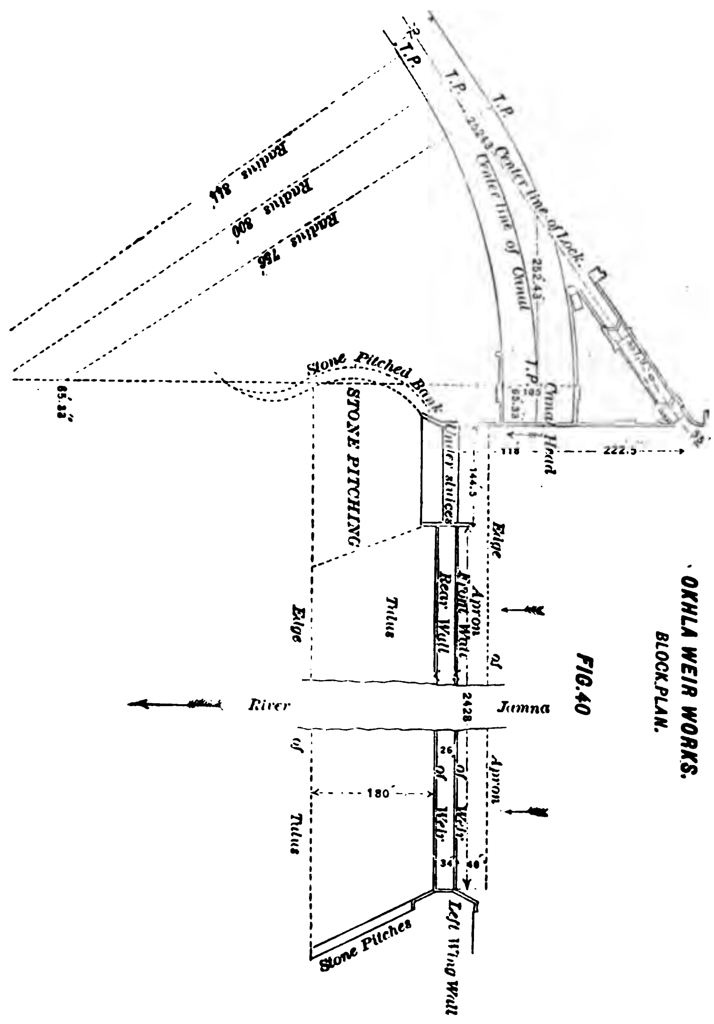


FIG. 40

OKHLA WEIR WORKS.
BLOCK PLAN.

stream edge. During the next winter, 1871–72, the embankments were heightened and strengthened; and

1,000,000 cubic feet of stone were expended in filling up the holes below the talus. In 1872–73, a second wall—the true crest wall of the weir, parallel to, and thirty feet above the one first built—was raised to a height of nine feet; the interval between the two walls being filled with dry rubble. A third wall, four feet thick and four feet deep, was inserted in the talus, forty feet below the lower wall; this has quite stopped all movement in the upper part of the talus; this wall is at the line of maximum velocity in floods. In March, 1874, the canal was opened. The total quantity of stone in the weir is 4,660,000 cubic feet. The stone is the quartzite of the ridge of Delhi, and of similar outcropping ridges in the country around. The right flank of the Okhla weir abuts on to a ridge of this rock, which has furnished an inexhaustible supply of material on the spot. The stone contains a large proportion of quartz, a little feldspar, and protoxide of iron. It is very durable and excessively hard, rendering it unsuitable (owing to the labor and expense) for finely dressed ashlar work. The river bed has silted up to the crest level; but at the canal head a clear channel is kept open by the scouring action of the river sluices placed at the right end of the weir, similarly situated to those of the Narora Weir, as described below.

This weir, and also the Sone and Narora weirs, have long aprons of dry rubble, and this seems to be the section selected for the modern dams, in sandy rivers, by Indian engineers. As the sandy beds of these rivers, except in the vicinity of the scouring sluices, were always raised, on the up-stream side, to the level of the crest of the weir, consequently that portion of the work would be free from scour, and it, therefore, was given a steeper slope than the apron on the down-stream side.

The Streeviguntum Weir or Anicut, over the Tambrapoorney River in Madras, is shown in cross-section

in Figure 41. This weir, and also that across the Godavery, Figure 44, are located in Madras, and they are there called *anicuts*. The Streeviguntum Weir is of the same type as the Narora Weir, though on a smaller scale.

STREEVIGUNTUM ANICUT

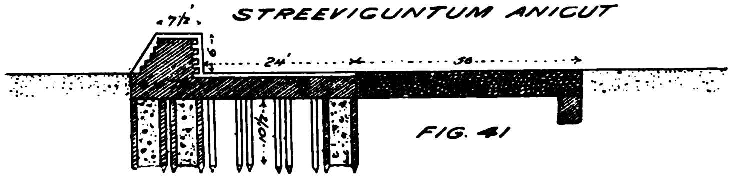


FIG. 41

It, however, has no water cushion, while the Norora Weir has a water cushion of at least three feet at the low stage of the river. The scouring sluices of this weir, as in the greater number of the old weirs of the Madras Presidency, have a small span. In this case there are nine vents, and each vent is only four feet in width by nine feet high. These small vents have not the scouring capacity of the large ones of Northern India.

The Streeviguntum Weir is 1,380 feet in length between the wing-walls, raised six feet above the average level of the deep bed of the river, and the width at the crown is seven and one-half feet; there is a front slope of one-half to one, and in rear a perpendicular fall on to a cut-stone apron twenty-four feet wide, and four and one-half feet in depth; beyond, there is a rough stone talus of the same depth, and thirty-six feet in width, protected by a retaining wall. The foundation of the body of the work, and of the cut-stone floor in rear, is of brick-in-lime, laid on wells sunk ten and one-half feet in the sand, and raised four and one-half feet above the wells, including the cut-stone covering; the retaining wall is built of stone-in-lime, and rests on a line of wells, sunk to the same depth, ten and one-half feet. The body of the anicut is of brick-in-lime, faced through-

out with cut stone, and furnished with a set of under-sluices at each extremity of the work, to let off sand and surplus water.*

The Narora Weir, Lower Ganges Canal in India, is shown in cross-section in Figure 43. This canal gets its supply from the river Ganges. It is the most recent of the large and important weirs, built across wide rivers with sandy beds, and from the volume of the floods, the sandy nature of the river bed, and the absence of material on the site suitable for a weir of this description, the difficulties to be contended with have been very great. The dam proper is a solid wall of brick masonry 3,700 feet in length: the floor below is of concrete, three feet thick, covered over with brick work, one foot thick, and then with one foot of sandstone ashlar: and the talus below is formed of very large masses of block kunkur, a kind of nodular limestone, brought from the quarries at thirty miles distance. The up-stream side of the weir is backed with clay puddle, pitched on its outer slope with an apron of block kunker.

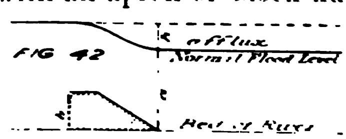


FIG 42

The length of the weir was settled by Major Jeffreys, R. E., as 4,000 feet, on the following data: See Figure 42.

h = afflux = 1.5 feet, when the river was at its highest was accepted as perfectly safe.

[illegible] 6 feet = maximum flood level above sill of weir.

Q = maximum flood volume flowing over weir of 200,000 cubic feet per second.

[illegible] length of crest of weir in feet.

6 feet per second = [illegible] velocity of approach.

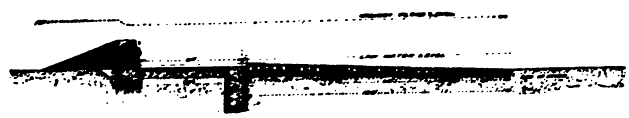
* [illegible] Weirs, by Major A. M. Lang. Professional Papers on Indian Engineering, Vol. VI, Second Series.

The figures applied in D'Aubinson's formula give:—

$$Q = 3.49\, l\, h \sqrt{h + .035\, v^2} \quad 4.97\, l\, (a - b) \sqrt{h + .01\, v^2}$$

Computing this we find the value of l = length of weir = 3,776 feet.

FIG. 43. NARORA WEIR, LOWER GANGES CANAL.



Colonel Brownlow, R. E., in reviewing the project and deprecating a proposed reduction of the length settled by Major Jeffreys, showed that a maximum flood of 230,000 cubic feet might not unreasonably be expected, and that taking into consideration the circumstances of the site, the light and friable nature of the soil of the country, and the lowness of the ridge which intervenes between the present channel of the river and the broad parallel trough of the Mahewah Valley, it would be very dangerous to contract the weir and raise flood levels.

The necessity for well foundations, especially for a strong line of deep blocks, along the lower end of the stone floor, and also for staunching all leakage by a puddle of clay above the drop wall—with a view of holding up all the water possible, and thus losing none of the supply when the river is at its lowest—of stopping all flow under the floor to the risk of undermining and destroying it—and also of resisting retrogressive action below the weir, was strongly urged in Colonel Brownlow's review of the project, as will be seen from the following extract from his report:—

"My reasons are, first, that all our experience in Upper India shows that where velocity of a stream is largely augmented by the construction of a barrier across it, permanent deepening of the channel below invariably takes place: and secondly, that leakage will

occur through the sandy bed underneath a dam with shallow foundations.

"Deepening of the bed has taken place on all the torrents across which weirs have been thrown on the Eastern Jumna Canal, and it is now occurring at Okhla. It occurred below the Dhanowrie Dam, on the Ganges Canal, until the obstruction caused by the dam was reduced, so that the normal velocity of the torrent was nearly restored, when the channel below partially silted up again.

"This fact alone is a very strong argument against the proposed reduction of length of weir, but as our weir at Narora will, in any case, greatly accelerate the mean velocity of the floods, we must be prepared both for retrogression of levels, and the formation of very deep holes immediately below the talus of heavy material. Those at the tail of the Okhla Weir, Figure 39, after the floods of last season, were from 19 to 20 feet deep; but whereas at Okhla the materials for filling them up, and thus resisting further retrogression, are readily available, we shall at Narora have nothing but a scanty supply of block kunkur brought from long distances, or blocks of béton manufactured at considerable expense.

"In the latter case, a strong line of deep blocks, supported by the ruins of the talus, would stoutly resist any retrogressive action, whilst the materials for repair were being collected and prepared; while the work on shallow foundations would run the greatest risk of being undermined and destroyed.

"It is stated that the leakage, prevented by deep well foundations, is more imaginary than real, because long before the volume entering the canal is likely to be utilized, the bed of the river will have become silted up nearly to the crest of the dam, and the upper layers of silt will have become more or less clayey, because leak-

age takes place through the banks as well as through the bed, and finally because little or no leakage has been detected through the Okhla weir which has shallow foundations.

"I cannot admit that the upper layers of silt deposited in the bed of the river above are deposited by falling floods, and are swept out again by the full current of the next succeeding high flood. The scour which takes place immediately *above* any marked contraction of a stream, is a matter of common experience, and is easily explained by the great relative increase in the bottom velocity resulting from the contraction.

"The banks, on the contrary, will become permanent, if the flanks of the weir are not turned, and they may ultimately become staunched by the clay brought down by the flood water. Besides, the effect of the pressure of the water on the banks is not worth mentioning, when compared to that on the sand underlying the weir. I think, therefore, that any consideration of the leakage through the banks may safely be neglected. But, even if it could not be, I do not see why we should not try and stop the leakage through the bed, because the banks are supposed likely to leak also.

"The latter argument applies equally to the objection commonly urged against deep block foundations, viz: that a line of them cannot be made perfectly water-tight. It is surely better to block up $\frac{99}{100}$ths of the area through which leakage can occur, than to leave it all open because a perfectly water-tight partition cannot be made.

"Apart from any consideration of the value in money of the water saved by a strong water-tight dam, the strongest necessity is, to my mind, laid upon us to economize every drop of the low-water supply in the river, owing to its insufficiency for the requirements of the years of drought. Common justice to the cultivating

community, dependent on the canal, seems to me to dictate the adoption of every reasonable precaution for rendering the whole of the short supply available for purposes of irrigation.

"I have placed the deep line of blocks at the tail of the cut-stone apron, because I think that the latter, *if built at the proper level, and of a proper section*, will perfectly protect the blocks from any fear of action on the up-stream side, and that the real danger to be guarded against is the cutting back and permanent deepening of the bed of the river below the weir. I have allowed only shallow foundations for the drop wall, because I consider the line of blocks underneath it sufficiently protected by the cut-stone apron and deep foundations on the down-stream side, and by the mass of heavy material on the up-stream side. The velocity of the current above the weir, although amply sufficient to sweep away the loose sand of the bed, has been proved, by the experience at Okhla, insufficient to move the heavy material of the apron."

To hold the talus together, it is traversed from end to end by solid concrete walls at intervals of thirty feet and forty feet as shown in Figure 43. This plan was found to be necessary at Okhla, Figure 39, where the third wall of four feet square section was adopted as necessary, in order to check the movement in the blocks in the upper part of the talus, although it formed no part of the original design.

The level of the cut-stone floor of the weir, as also of the floor of the under-sluice, is three feet below low water level; and as the floor is five feet thick, the laying of it entailed excavation to a depth of eight feet below low water. To effect this, the upper row of blocks and lower row of wells were sunk to full depth, and hearted with concrete. This was done by filling the hole below the curb, and

the lower one or two feet of the block or well by hydraulic cement let down in skips; when this had set, it formed a water-tight plug, and enabled the well or block to be pumped dry. The concrete core of the well or block was then put down in layers, and rammed in the ordinary manner. The interval between each pair of contiguous wells and blocks was closed by wooden piles, and the interval, included between piles and well, cleared of sand and filled with concrete. Clay puddle was also packed above the upper row of blocks. The space, thirty-three feet in width intervening between the upper row of blocks and the lower line of wells, was then divided into compartments of about forty feet in length, by cross lines of shallow blocks, sunk, hearted and connected as above described. Thus large coffer-dams were formed, which were excavated to a depth of eight feet below low water level, and the water pumped out by Gwynne's pumps, so as to allow of a three feet thick concrete floor being laid. On this a layer of brick-work one foot thick was added; and this in its turn was covered by an ashlar floor of cut sandstone blocks one foot in thickness.

The under-sluices, forty-two vents of seven feet each, are at the extreme right abutment end of the weir, so as to keep a clear channel open along the front of the immediately adjoining head-sluices of the canal, whose floor is three feet above that of the weir sluices, and this allows the lowest three feet of silt-laden water to pass by without entering the canal. The crest of the weir stands seven feet above low water level, which is the level of the floor of the head-sluices, thus allowing seven feet in depth of water to pass into the canal.*

The Dowlaiswaram Branch of the Godavery Anicut or Weir is shown in cross-section, in Figure 44.

Indian Weirs, by Major A. M. Lang. Professional Papers on Indian Engineering, Vol. VI, Second Series.

The total length between the extreme flanks of the weir is 20,570 feet. It is broken into four sections separated by islands, and the total length of the anicut on these four sections is 11,866 feet.

SECTION

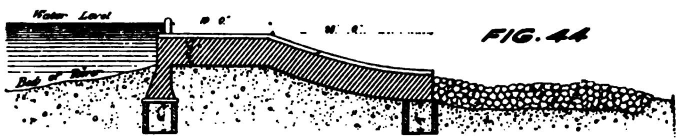


FIG. 44

GODAVERY ANICUT

The longest section is the Dowlaiswaram, and the following description of this section is taken from Colonel Baird Smith's work, "Irrigation in Southern India:"

The bed of the Godavery throughout is of pure sand, and in such soil are the whole of the foundations laid. Commencing from the eastern or left bank, the first portion of the work is the Dowlaiswaram branch anicut or dam. The total length of this is 4,872 feet. The body of the dam consists of a mass of masonry resting on front and rear rows of wells, each well being six feet in diameter, and sunk six feet below the deep bed of the stream. The masonry forming the body is composed:—

1st. Of a front curtain wall running along the whole length, seven feet in height, four feet in thickness at the base, with footings one foot broad on each side to cover the tops of the wells on which the curtain wall rests, and three feet thick at the summit.

2d. Of a horizontal flooring or waste-board nineteen feet in breadth and four feet in thickness.

3d. Of a masonry counter-arched fall twenty-eight feet in breadth and four feet thick, of which the curve is so slight that the form may be considered practically as that of an inclined plane. The waste-board and tail

slope are protected against the action of the stream by a covering of strongly clamped cut stones over all.

4th. Of a rough stone apron in rear formed of the most massive stones procurable, and extending about seventy or eighty feet down stream. Figure 44 does not show the apron extended so far, but it is now extended to about 150 feet, and further secured by a masonry bar.

The apron protects the rear foundation against the erosive action of the stream passing over the dam. The body of the dam rests merely on a raised interior or core of the *common river sand*, and no precautions to strengthen this in anyway have been considered necessary. On the extreme left flank of the dam is a series of works, consisting of a lock for the passage of craft, a head sluice for an irrigation channel, and an under sluice for purposes of scour and clearance from deposits.

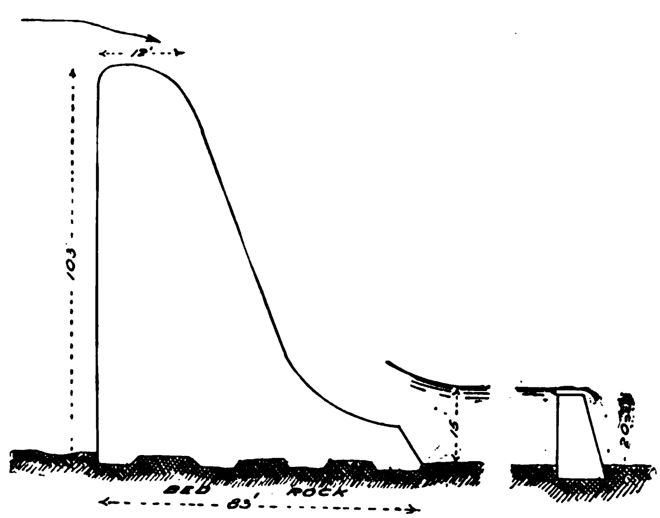


FIG. 45. CROSS-SECTION OF TURLOCK WEIR.

The Turlock Weir in California, across the Tuolumne River, is shown in cross-section, in Figure 45. The

original design for this weir was made by Mr. Luther Wagoner, C. E., but the water cushion was added by Mr. E. H. Barton, the present Chief Engineer of the Turlock Irrigation District. The flood discharge of the river will pass over the weir. The low weir on the down-stream side backs up the water and forms a water cushion to receive and break the shock of the flood-water when it flows over the weir. The water-cushion has been found in India to be a most effective protection to the bed of the river from the erosion caused by falling water.

The length of the weir on top is 330 feet, its maximum height to foundation 108 feet, and the maximum height of the overfall of water ninety-eight feet. Its width at base is eighty-three feet, and the maximum pressure 6.3 tons per square foot. The weir is curved in plan, the radius to up stream face being 300 feet, and the angle 60°. Bed and sides of channel is metamorphic (quartzite after slate) rock of exceeding hardness.

On the removal of an old dam, near the site of this weir, an inspection of the bed rock, where the fall had been ten to thirty feet over said dam for eighteen years, showed hardly any appreciable wear. Calculated for the highest flood known, that of 1862, the flow over the crest of the weir is 130,000 cubic feet per second.

The subsidiary weir is located 200 feet below the main weir. It is 120 feet long on top, twelve feet in width, and twenty feet in maximum height, and it backs the water to a depth of fifteen feet on the toe of the upper or main weir, giving a water-cushion of that depth; but, during floods, there will be a depth on the toe of over forty-five feet. The volume of the dam will be about 33,000 cubic yards.

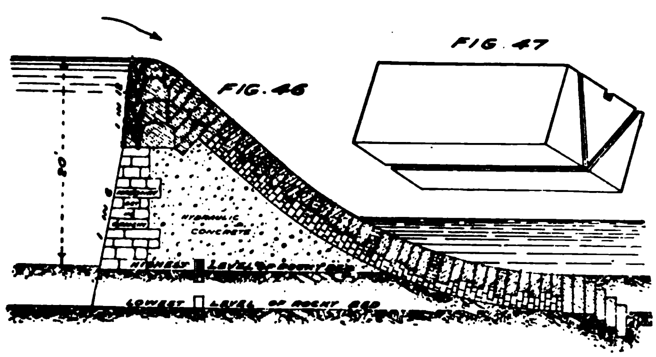
Vertical falls, with water-cushions, are preferred in India to sloping or curved faces on the down-stream side

of works over which water falls. For stability to resist water pressure dams or weirs, with a curved profile according to the French plan, are the most suitable, but, in order to avoid the erosive and destructive action of the falling water on the curved face, vertical walls with water-cushions are preferred.

Numerous instances of vertical falls are to be found on the canals in Northern India, and on two important modern works, the Bhim Tal Dam and the Betwa Weir. Doubtless instances can be found also in modern works of dams with curved faces on the down-stream side over which water flows—for example, the Vrynwy Dam for the Liverpool water supply, and the concrete dam for the Geelong, Australia, water supply.

The dam across the river Lozoya, in Spain, to impound water for the supply of Madrid, has an extraordinary vertical drop—105 feet. The back of the dam, over which the water falls, is not vertical, but has a slight batter given to it by off-sets. The flood-water, however, leaps clear over this face.

During floods, when the reservoir is full, the whole discharge of the river pours over it in an unbroken



WEIR OF HENAPES CANAL.

sheet. It has not a water-cushion. The dam was built of ashlar, which is not the best method of construction

for such a work. Uncoursed rubble is much better suited for a dam, as there is less likelihood of percolation through its broken joints than through the regular-coursed ashlar.

The cross-section of the weir of the Henares Canal, on the river Henares, in Spain, is given in Figure 46. The masonry of this weir is first-class in every respect. Its design, however, as to its cross-section, is one not adopted in India. For a masonry weir, a vertical drop on the down-stream side and a water-cushion, is preferred in the latter country.

The action of the water on the Ogee Falls, on the Ganges Canal, was found very destructive, whereas the vertical falls with water-cushion stood well.

Where the Henares Canal is taken from the river, the river bed is composed of compact clay rock, mixed with strata of hard conglomerate, which had to be blasted out to fit it for the foundation of the weir. The weir itself is 390 feet in length of crest, formed on two curves of 397 and 198.5 feet, running obliquely across the river so as to be tangential to the axis of the canal. It raises the water to a height of twenty feet. Its thickness at crest is 3.14 feet, and on the general level of the river's bed, 45.8 feet. As this bed, however, was very uneven, it was necessary to carry down the thrust of the apron by a series of blocks of stones formed in steps, the last firmly embedded three feet in the rock. The body of the weir consists of hydraulic concrete; the apron is faced with cut-stone blocks, every fifth course being a bond three feet deep, and is a beautiful specimen of masonry. Much pains have been bestowed on preventing the least filtration. For this purpose a channel was cut in the rock along the central axis of the weir for its whole length, and into this a line of stones was fitted, half bedded in the rock, half rising into the concrete.

Into each vertical joint of these stones a groove was cut an inch deep. The stones were built in cement into the rock, and the joints run with pure cement. The concrete was then rammed tightly round them, and a water-tight joint thus formed.

With the same object V-shaped grooves were formed in the sides of each stone of the four upper courses of the weir, as shown in Figure 46, and horizontal grooves cut to correspond with them on the upper and lower faces of each stone, as shown in Figure 47. When, therefore, the stones were set, there was formed a continuous channel, one inch square, running between each, and this was filled with pure cement, poured in liquid, so as to form a tight joint between each stone.

In spite of all the precautions taken the floods exerted an erosive action on the bed of the river *below* the weir, and a large hole was scooped out of the rock at the tail, where the apron ended.*

The head of the Cavour Canal in Italy is on the left bank of the Po, about a quarter of a mile below the Chivasso bridge.

The bird's eye view, Figure 48, shows the position of the weir, regulator, under-sluices and escapes. In Indian canals the escapes are usually channels from the main canals to carry away any surplus water. In the bird's eye view the channels marked "Escapes" are really channels to carry away the water that is used for scouring purposes. These scouring or under-sluices were intended to prevent the silting up of the channel from the left bank of the Po to the regulator.

In the bird's eye view is shown the location of the proposed weir (not yet built), placed obliquely in a curve, across the river.

**Irrigation in Southern Europe*, by Lieut. (now General) C. C. Scott Moncrieff, R. E.

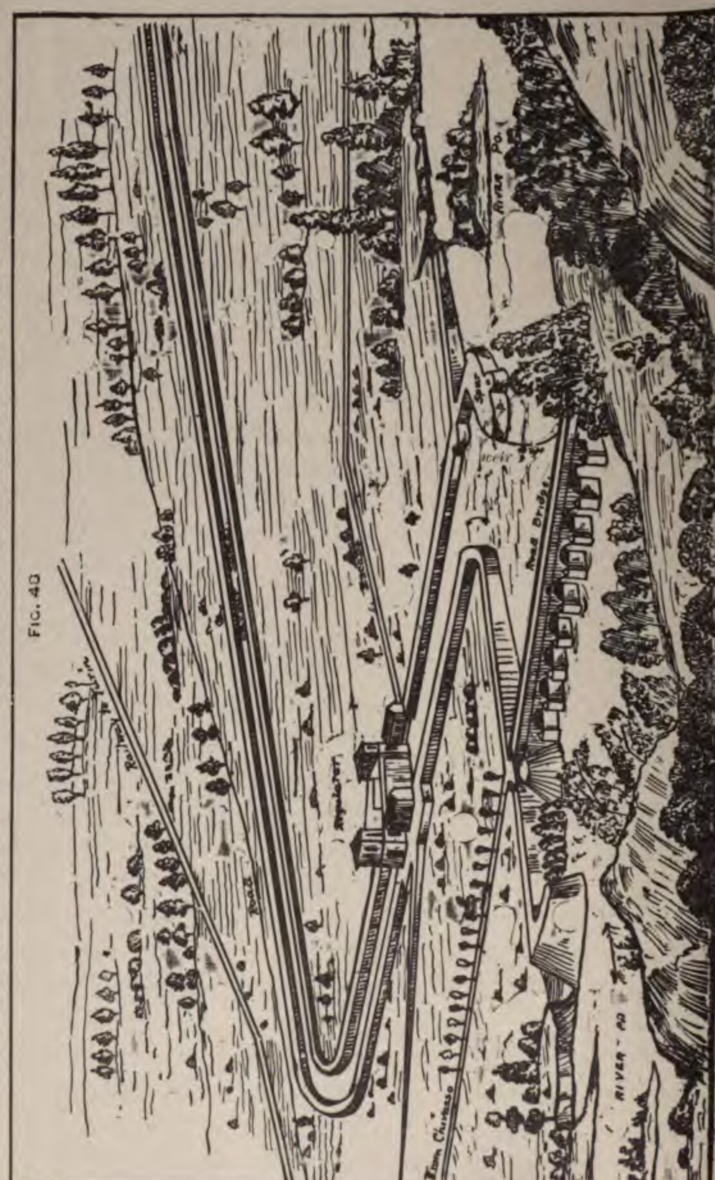


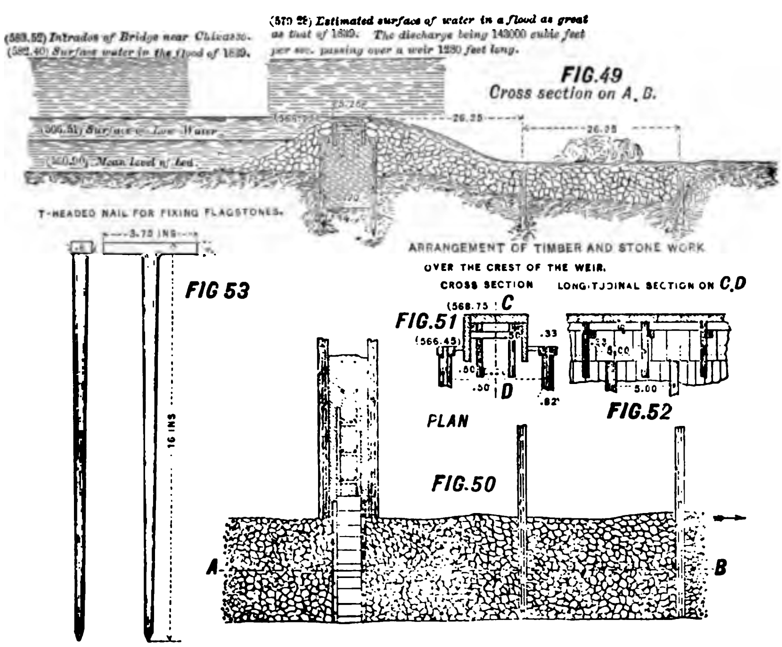
FIG. 40

BIRD'S-EYE VIEW OF SITE OF HEAD WORKS CAVOUR CANAL.

Its length was to be 2,300 feet, forming a curve of, for the most part, 823 feet radius, but less at the ends. The design was to raise the water by means of this weir to a height of about eight feet; and of such excellent stiff soil is the bed composed, that it was thought sufficient to build a wall of concrete going down to only

CAVOUR CANAL—DETAILS OF PROPOSED WEIR ACROSS THE PO.

THE FIGURES IN BRACKETS GIVE THE LEVEL ABOVE THE SEA IN FEET.



6.56 feet below the bed, enclosed in front and rear by sheet piling, its upper portion cased in granite slabs of five inches in thickness, and the rest of blocks of rough stone forming a protection in front sloping down to a horizontal distance of sixteen feet, and in rear to twenty-six feet, with a line of sheet piling at its toe, and beyond it an apron of similar blocks, of the same width of twenty-six feet, with another row of piling.

The weir was intended to rest on solid abutments at the two ends, and on the end next the canal was to be supplied with a set of scouring sluices, or escapes, consisting of seventeen openings, each 4.6 feet in width and eight feet in height. All the bed for ninety-six feet below, and 500 feet above, is to be paved with splendid blocks of cut granite, brought from the neighborhood of the Lago Maggiore.

From the left flank of this escape the regulating bridge is retired for a distance of about 700 feet, as shown in the bird's-eye view, Figure 48, and close to its right abutment is built a second escape of nine openings, each 5.54 feet wide and ten feet high. The floor of this escape is one foot lower than that of the regulating bridge, the more effectually to establish a scour.*

Article 29. Scouring Sluices—Under Sluices.

These sluices are sometimes called Weir Sluices and again Dam Sluices.

The first effect of the construction of a weir across a river is that the pool formed by it gradually silts up, partly by deposit, during floods, of matter in suspension in the water, and partly by the gradual forward motion of the bed of the river which exists in all streams, but is only visible to the eye in rivers with sandy beds. Islands begin to form, which would in time obstruct navigation across the river above the weir, and would prevent the water, in the dry season, from finding access to the canals led off from the pool.

In rivers in India carrying sand and silt, the silting up of the bed of the river to the level of the crest of the weir seems to be inevitable. This sand and silt if not

*Irrigation in Southern Europe, by Lieut. (now General) C. C. Scott Moncrieff, R. E.

removed choked the head of the canal and locks, located above the weir, and stopped their supply of water. In order to obviate these difficulties, every weir has to be furnished at one extremity, or at both extremities, according as one canal or two canals are taken off from above it, with a set of scouring sluices. In very long weirs, such as that across the Sone River in Bengal, another scouring sluice is placed in the center, to assist in keeping open a navigable channel across the river.

The proper location for a scouring sluice, with respect to the regulator of a canal, is that the crest of the weir should be at right angles to the face line of the regulator, and also at right angles to the face line of the lock when the channel is navigable.

This is well exemplified in the plan of the Okhla Weir Works, Figure 40. It can be seen there that the current of the river flows flush with, and parallel to, the face line of the canal and lock, and that there is no recess for still water and consequent silting.

An instance of the defective location of a regulator, with respect to the scouring sluices, is seen at the headworks of the Upper Ganges Canal, Figures 26 and 28. Here, the entrance to the canal is located over two hundred feet lower down the river than the scouring sluices, in consequence of which, the strong current flowing to the scouring sluices, is *not across* the face of the regulator, and there is a tendency to silt where the marks * * * are shown, on Figure 28. In the low stages of the river the silt prevents the free flow of the current towards the regulator.

Another instance is in the location of the regulator of the Cavour Canal, Figure 48. The regulating bridge is located 700 feet below the weir, and the course of the current through the scouring sluices, leaves somewhat still water in the left corner, just above the sluices of

FIG. 54. MYAPORE REGULATING BRIDGE.

the regulator. When these sluices are opened the silt is washed into the canal.

The flow through the scouring sluices is controlled as explained in Article 31.

The regulator should be as close as possible to the scouring sluice, and the sill of the latter should be about three feet lower than the sill of the former. Under these circumstances, when the scouring sluices are opened, the scour takes place across the whole face of the regulator, and washes away any silt or debris likely to obstruct the free flow of the supply into the canal.

Article 30. Regulators.

The Regulator at the head of a canal is also called, Regulating Bridge, Regulating Gate, Regulating Sluice, Head Gate, Head Sluice, Canal Sluice, Head of Canal, etc. To be precise, the regulator is the structure in which are fixed the sluice gates to control the water supply to the canal.

Regulating Gates Del Norte Canal—Cross-Section and Elevation.

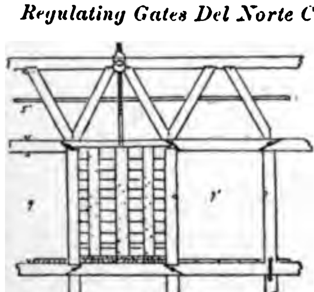


FIG. 55

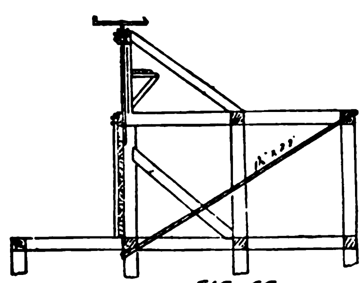


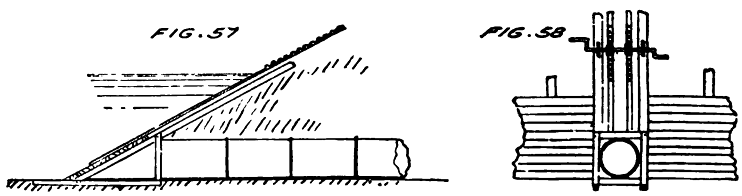
FIG. 56

In India, Egypt and Italy, the regulator is sometimes part of a highway bridge, as shown in Figures 27, 32 and 48. In the latter case, however, the use of the covered bridge is confined to the canal officials.

The Myapore Regulating Bridge, of the Upper Ganges

Canal, is shown in Figure 54. The view is taken from the down-stream end of the bridge, so that the sluice-gates, which are located on the up-stream side, are not seen. The sluice-gates of this regulator are shown in Figures 62 and 63. These sluice-gates are twenty feet in width, but regulating sluices are seldom more than six feet in width, on account of the difficulty of working large sluices under a great head of water.

Idaho Canal Regulator Head—Cross-Section and Elevation.



The Reglating Gates of the Del Norte Canal, in Colorado, are shown in Figures 55 and 56, and Figures 57 and 58 show an Idaho Canal Regulator Head for pipe inlets.*

It is usual, on Indian canals, to make the floor of the regulator at head of canal, the zero for levels on the canal.

Article 31. Sluices—Gates—Movable Dams and Shutters.

The terms Sluices, Head Sluices, Gates, Sluice Gates and Head Gates, are variously employed to mean the sluices that are fixed in the regulator, and which are used to control the supply of water at the head of a canal. The sluice gates usually used on Irrigation Canals are the sliding sluice gate Figures 55, 56, 57, 58, 59, 60,

*Figures 55, 56, 57 and 58, are taken from a paper on American Irrigation Engineering, by Mr. H. M. Wilson, M. Am. Soc. C. E., in Vol. 24 of the Transactions of Am. Soc. C. E.

62, 63, 64, 65, 69, 70 and 71, the horizontal plank gate, Figures 20 and 61, and the vertical plank or *needle* sluice. These, and other methods not so generally in use, are explained below.

The gates of the Cavour Canal Regulator, of which there are twenty-one, are fitted with a double set of gates, and the cutwaters of the piers, on the up-stream

SLUICE GATE—CAVOUR CANAL.

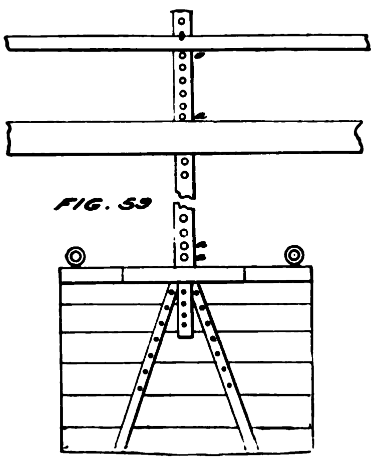


FIG. 59

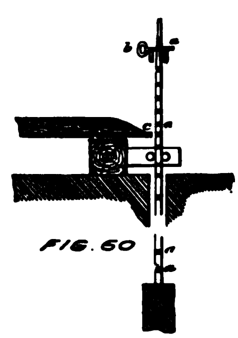


FIG. 60

SLUICE GATES—INDIAN CANALS.

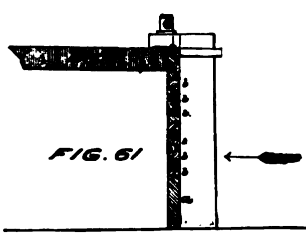


FIG. 61

Old Method on the Jumna.

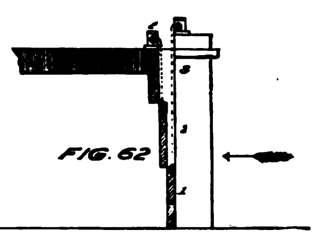


FIG. 62

Improved Method on the Ganges.

side, have grooves besides, for stop-planks. These latter are intended to be used in case of accident or repairs to the gates or regulator.

The gates are of wood, braced with iron, as shown in Figures 59 and 60. They are raised by means of an iron bar four by three-quarter inches, and about eighteen feet long, firmly fastened to the center of the upper edge of the gate, and connected with diagonal bars to the lower corners to distribute the force. This bar passes through, to the platform above the highest flood level, from which the gates are worked. The bar is pierced with holes, *a*, *a*, one and one-half inches in diameter at every two inches of its length, through which, when it is required to raise it, the iron point, *c*, of a crow bar is put, and it is raised up hole by hole, an iron key, *b*, being pressed at the same time through another of these holes, and resting on two cross bars to prevent it slipping down again. One man works the crow-bar while another holds the key. By pulling this out the gate falls at once, and this is important, as it is of consequence sometimes to be able to close the canal quickly.

This arrangement has the great merit of simplicity and it is frequently adopted on American canals.

On some of the regulators in Northern India, a drop gate is used in a simple groove, and sleepers, with a scantling of six inches square, are dropped upon the top of the gate. Both time and labor are required to close or open the bays, although they were only six feet in width. On the Ganges Canal regulator, however, with ten bays, having each a width of twenty feet, on which the safety of the works depended, it was necessary to devise some quicker method to economize the labor required for using the apparatus. Figures 61 and 62, show the old and also the improved method of operating the sluices. Figure 61 represents, in section, the drop gate, *a*, and the sleepers, *b*, *b*, *b*, opposed to the up stream current; *a*, represents a gate five feet in depth, which is kept suspended in dry seasons, and is dropped down on the expectation

of a flood; *b*, *b*, *b*, show the sleepers or long bars of timber, which when the chains are removed from the gates, are successively dropped upon them until the bay is

CANAL REGULATING APPARATUS.

REGULATING BRIDGE WITH LIFT-GATE & SLEEPERS

Elevation.

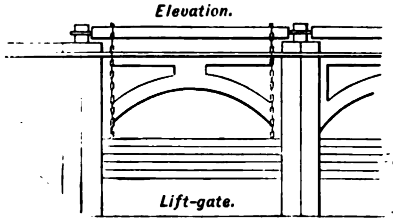


FIG.63

DROP-GATE FOR DAMS.

Elevation.

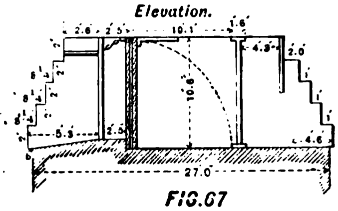


FIG.67

Plan

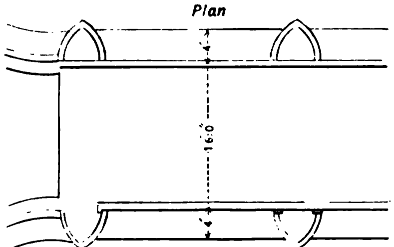


FIG.64

Plan

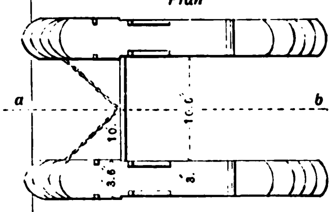


FIG.68

WINDLASS FOR REGULATING BRIDGE.

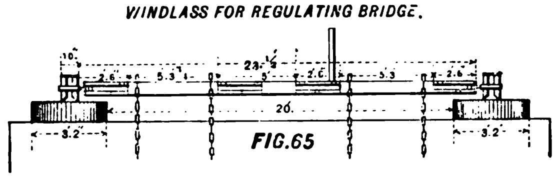


FIG.65

PLAN OF SLEEPER.

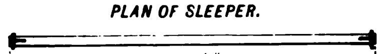


FIG.66

closed, Figure 61. The time that this takes is equal to eighteen minutes.

Figures 62, 63, 64 and 65, show the improved design gained by the use of the windlasses. The bay or sluice opening, it will be observed, is divided into three series, the lowest shutter having its sill on the floor of the regulator, which is the zero level of the canal; the centrical and top shutters having their sills elevated in heights of six feet, but working in separate grooves in the piers. The shutter marked 1, Figure 62, is dropped from windlass, 1; that marked 2 from windlass, 2; and that marked 3 consists of sleepers, which are raised and lowered without the aid of a windlass. The three gates, therefore, are quite independent of each other; each has its own sill to rest upon; and the whole can, if necessary, be worked simultaneously. The great advantage of this method will be understood, by supposing that a supply of water not exceeding six feet in depth is required for canal purposes. In this case, the whole of the shutters, 2, and 3, may remain closed; and when floods come on, the whole of the water-way may be stopped by releasing one gate only.

The machinery attached to these gates is of the most simple description, intelligible to the commonest laborer on the works, and not liable to disarrangement.

On some canals in India and also in other countries, Needle Dams, as they are termed, are adopted to control the supply. A horizontal bar of wood or masonry is fixed on the floor of, and across the opening, and a beam of timber is placed vertically over and parallel to this and fitting into sockets in the piers. Planks, called needles, about four inches scantling, are placed vertically in front of these, and are operated from the flooring of the bridge, whether permanent or temporary. This plan has been found to work well in some places. There is always

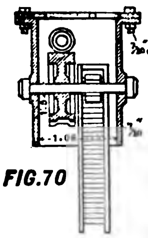


FIG.70

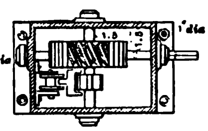


FIG.71

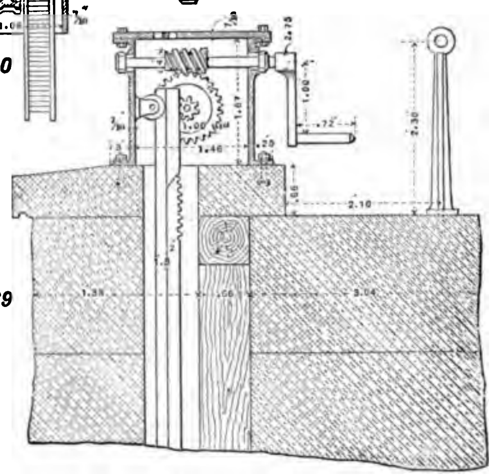
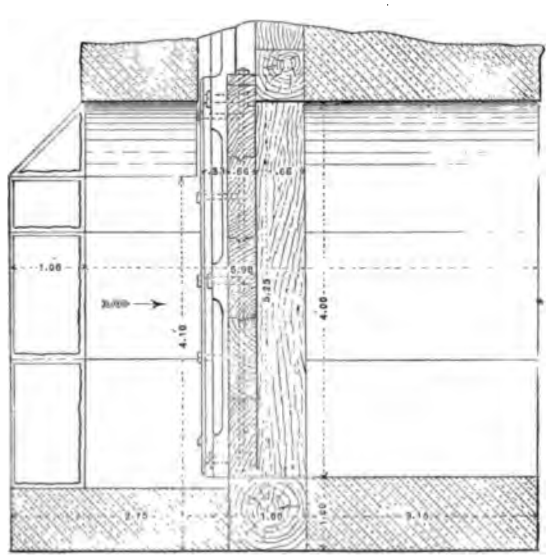


FIG.69



SLUICE OF HENARES CANAL

more leakage through a plank sluice than through a properly constructed framed sliding sluice.

For the openings of *Level Crossings*, *Drop Gates* are sometimes provided. They are retained in their upright position, Figures 67 and 68, by chains against the pressure of the canal water from the inside, and which, on the occurrence of a flood, can be dropped down on the flooring by releasing a catch, and allowing the flood water to pass through the openings. When the flood is over, the gates are raised upright by a movable windlass, the pressure of the water being temporarily taken off by dropping planks into the groves.

The sluices of the Henares Canal are five in number, each four feet wide. The details of these sluices are shown in Figures 69, 70 and 71.

The gates are made of elmwood, and rest, on their down stream side, against pinewood frames, instead of against the edges of the stone grooves, and thus considerably reducing the friction, and at the same time securing a tight fit and preventing loss of water by leakage.

The gates are raised by ratchets. One man can with tolerable ease raise a gate at a time. The ratchets, pinions, etc., are enclosed in rather heavy cast-iron boxes. This allows of no provision for suddenly dropping the gates in case of floods; but an overfall weir has been built in the left bank of the canal just below, to allow of any flood water above the full supply falling back directly again into the river.

What seems a strange omission is, that the piers of the regulator are not provided with grooves for stop-planks to be used to dam out the water in case of accident or repairs.*

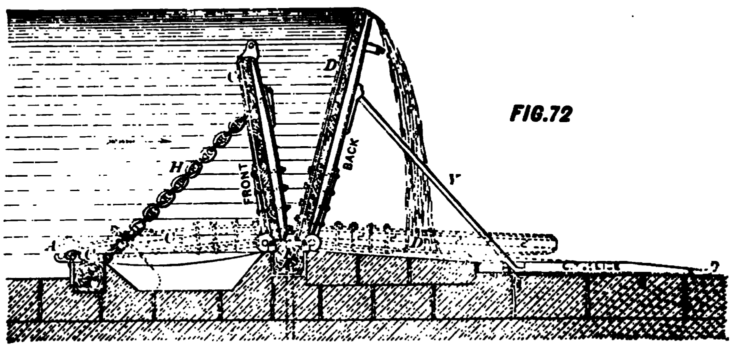
*Lieut. (now General) C. C. Scott Moncrieff, *Irrigation in Southern Europe*.

In the old works such as the Godavery, the Kistna, the Cauvery, and other weirs, it was the custom to make sluices with vents only six feet wide, and raised to only about half the height of the flood. In these works the scouring sluices were closed in the dry season, either by balks of timber dropped one after another into the grooves in the pier, or by gates, sliding in vertical grooves, which gates were raised and lowered from above by levers working into long rods attached to the gates. This system necessitated the construction of a masonry superstructure to above the level of the highest flood, which opposed great resistance to the free flow of the floods, and stopped floating *débris* in the river, so that the sluices not unfrequently became choked with trees and brushwood.

As these earlier works were inefficient, in the more modern works, much larger openings have been left, and movable dams have been erected, with no superstructure above the level of the weir, so that floods pass without obstruction over the weirs to the depth, it may be, of eighteen or twenty feet.

That these movable dams may thoroughly perform their duty, it is necessary that they should be large and strongly constructed, and that they should be capable of being operated quickly. It was, therefore, attempted, in Orissa, India, to increase considerably the size of the sluice openings in the weir in the Mahanuddy River, and shutters on the plan adopted by the French engineers in the navigation of the Seine were constructed. The center sluices are divided into ten bays, of fifty feet each, by masonry piers. Each bay is composed of a double row of parallel timber shutters, which are fastened by wrought-iron bolts and hinges to a heavy beam of timber embedded in the masonry floor of the sluices. There are seven upper shutters and seven

lower in each bay. The lower shutters are nine feet in height above the floor, and the upper seven and a-half feet. Each bay is separated from the next one by a stone pier five feet thick, in which the gearing for working the shutters is fixed.



SHUTTERS OF THE MAHANUDDEE WEIR.

The up-stream shutter fell up-stream, and the down-stream shutter fell down stream, so that the up-stream shutter, unless intentionally fastened down, would, in times of flood, be raised by the water getting under it and flowing against it, and would thus, automatically, shut itself, and leave the shutter below quite dry. The up-stream shutter was supported up-stream by chain ties. When it became necessary to open the sluice again, of course it would not have been practicable to lower the upper shutter against the head of water standing against it, and, therefore, the lower shutter was raised and strutted up by hand, as men could walk about with safety on the dry, down stream sluice channel, and this left a double row of shutters standing against the stream. The space between these shutters was then allowed to fill with water, and then the upper shutter, being in equilibrium, was allowed to fall back into its place on the bottom of the sluice, while the lower shutter supported the head of water

If, at any time, it was required to open the sluice, the back shutter was lowered by knocking away the feet of the struts which supported it, on the down-stream side, and it then fell down-stream, and the sluice was open.

It has been found that in a dam constructed on this principle, 500 lineal feet of shutters can be easily lowered in one hour, with a head of six feet of water, and that with a similar head an equal length can be closed in twenty-five minutes and that three men (East Indians) standing on the floor are sufficient to knock away the back struts with safety to themselves. The back shutters are not damaged as they fall on the floor, because water escapes as each shutter falls, sufficient to form a a cushion for the other shutters to fall into. Twelve men are necessary to lift each of the back shutters into position.*

This kind of shutter has never been raised against a greater head of water than about six feet nine inches. The front shutter is only used when the level of the river has fallen to at least six feet above the floor of the weir, and frequently the engineers hesitate to use the shutters until the water has fallen lower.

The objection to this plan was, that the upper shutter was raised by the stream with such velocity and force that the chain ties supporting it frequently gave way, and the shutter was carried off its hinges. On one occasion the front beam was pulled up from the floor.

Major Allan Cunningham, R. E., has given the following formula, for finding the tension on the chains of shutters similar to those used on the Mahanuddy Weir.†

*Mr. R. B. Buckley, C. E., on *Movable Dams in Indian Weirs*, in Transactions of the Institution of Civil Engineers, Vol. 60.

†Professional Papers on Indian Engineering, Vol. 4, Second Series.

Let b = breadth of shutter in feet.

d = $\left.\begin{array}{l}\text{depth of shutter}\\ \text{" " stream}\end{array}\right\}$ in feet.

v = mid-surface velocity (over shutter when down) in feet per second.

w = weight of a cubic foot of water = 62.5 lbs.

g = acceleration of gravity = 32.2.

T = total sudden tension of the whole set of chains in lbs.

a = angle of inclination of chains to shutters when vertical, that is, at instant when strained taut.

Total tensile stress in lbs. $= T = \left(d + 4\frac{v^2}{g}\right) w\,b\,d$ cosec a.

Example. In the sluice shutters of the Midnapore Weir, given $b = 6'.25$, $d = 6'.5$, $v = 12'$ per second, $a = 55°$.

$$\text{Total tensile stress} = \left(6.5 + 4 \times \frac{12^2}{32.2}\right) \times 62.5 \times 6.25$$

$$\times 6.5 \times 1.221 = 75{,}587 \text{ lbs.}$$

And if there be two chains equidistant from the "center of percussion" of the shutter, then

Tensile stress of each chain = 37,794 lbs.

In order to diminish the violent shock caused by the rapid rising of the upper shutter, Mr. Fouracres, C. E., made important improvements in the method of working them. Figures 74, 75, 76 and 79, give four views of the shutters of the Sone weir in different positions. Figure 74 shows the sluice "all clear," with both shutters lying on the floor, the flood being supposed to be running freely between the piers, which are eight feet in height. When it becomes necessary to close the sluice and shut off the water flowing through it, a clutch worked from a handle from the top of the pier is turned, which frees the shutter from the floor, and it then floats par-

tially up from its own buoyancy, when the stream, impinging upon it, raises it to an upright position with great force, shutting up the sluice-way, Figure 75.

MIDNAPORE CANAL.
Tumbler Regulating Gear for Distributaries.

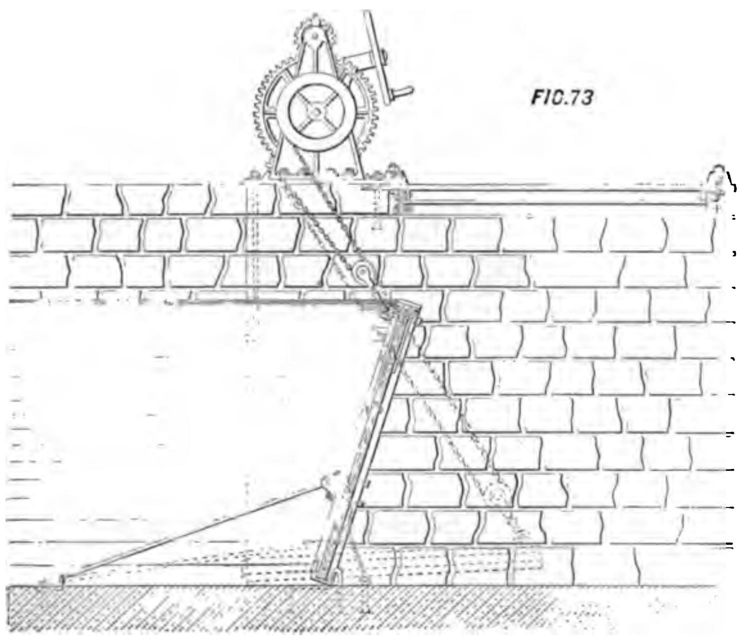


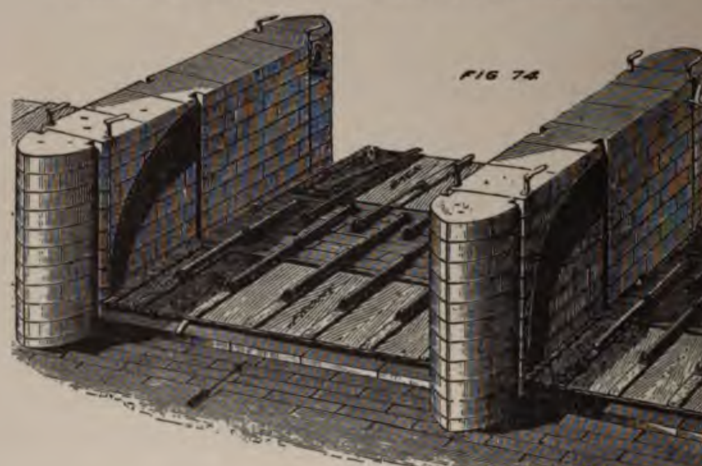
FIG. 73

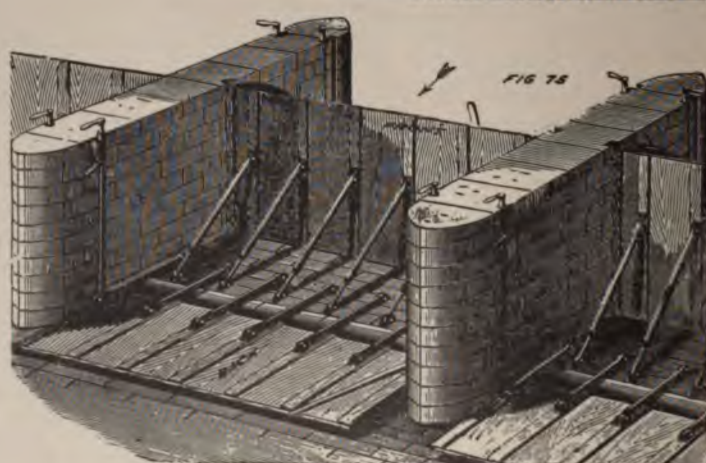
But if a shutter, twenty feet long and eight feet in height, were allowed to come up with such velocity, it would either carry away the piers or be carried away itself. To destroy this sudden shock, Mr. Fouracres fixed to the down-stream side of the upper shutters six hydraulic buffers or rams, which also act as struts for the shutters when in an upright position. These rams are simply pipes with a large plunger inside, as shown in longitudinal section, Figure 77, and cross-section, Figure 78.

The pipes fill with water when the shutter is laying

down, and when it commences to rise, the water has to be forced out of them by the plunger in its descent, and, as only a small orifice is provided for the escape of the

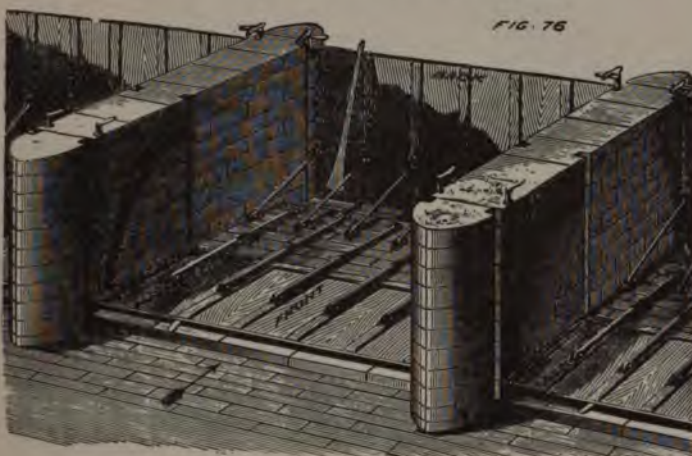
FOURACRES' SLUICES AT THE WEIR ON THE RIVER SONE.





water, the ascent of the shutter, forced up by the stream, is slow and gentle, instead of being violent. The orifices

in the pipes are covered with india-rubber discs to prevent them from being filled with sand or silt.



The water is now shut off effectually, as shown in Figure 75; but without other means being taken it would be impossible to open the sluice again, as it could not

FIG 77

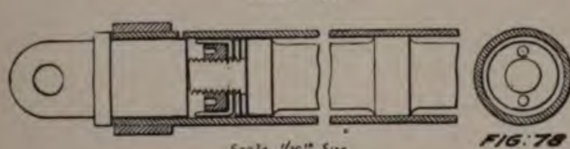


FIG: 78

SECTION OF HYDRAULIC BRAKE HEAD. SONE-WEIR.

be forced up-stream. The back shutter is therefore provided below it, as shown in this same view. This lower or back shutter is so arranged that it can be lifted up by hand and placed upright, ties being placed to support it, as shown in Figure 76. The water is then allowed to fill the space between the two shutters, and the upper one can then be thrown down on the floor again, but the lower one is held up by ties which are hinged to it at one-third of its height, and by this means it is "bal-

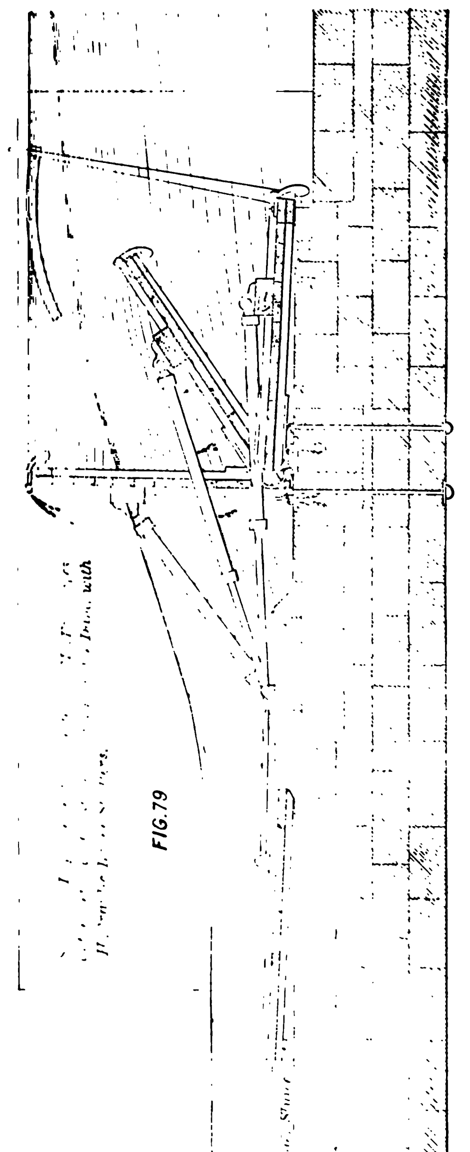


FIG. 79

CROSS SECTION THROUGH CENTRE OF PIER

anced," and resists the pressure on it until the water rises to its top edge, when it loses its equilibrium and falls over, thus opening the sluice again.

The sluices can be left to fall of themselves if the river rises in the night; or, if it is thought not expedient, they can be made fast by a clutch on the pier-head, as shown in Figures 74, 75 and 76. By these expedients, these large sluice-ways, twenty feet broad and eight feet deep, can be shut off or opened as required, with the greatest facility and expedition, and the whole set of twenty-five sluices can be opened in a few minutes, and when opened they can pass through them anything that the river brings down, without danger to the wier.*

Figure 79 shows the upper shutter as it is being raised by the current through the sluice-way.

During the hot season, when it is of importance to utilize all the available water, all leaks between the shutters and piers are calked with hemp and straw, to prevent loss of water by leakage.

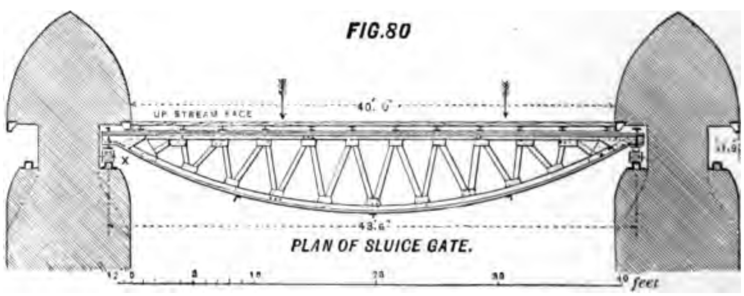
It has been found in practice on the Sone Weir, that the shutters can be safely lifted, without shock, against a head of ten feet of water, and they have been frequently worked under these conditions. The greatest head against which other shutters on any other weir have been lifted is believed to be about six feet nine inches only. It is stated to be a sight worth seeing to watch a stream of water twenty feet broad and eight or nine feet deep, flowing with a velocity of seventeen to twenty feet a second through the sluices, with a difference of ten feet between the water level above and below the sluices, to be suddenly closed by a single gate twenty feet long by ten feet deep. The water, when the shutter reaches the vertical, rises in a

*Engineering. September 10th, 1873.

wave one or two feet above the top of the shutters and piers, and flows over for a few seconds before it sinks to the mean level of the stream.

In the Transactions of the Institution of Civil Engineers, Vol. 60, Mr. F. M. G. Stoney, C. E., has given a design for a large span lifting sluice, shown in Figures 80, 81, 82, 83 and 84. This is inserted here to show a design that in certain circumstances it may be very advantageous to adopt, where it is necessary to have a large opening.

FIG. 80



PLAN OF SLUICE GATE.

The clear span was forty feet, and the depth of water twelve feet. The gates were designed to be lifted twelve feet. The up-stream face was vertical, and the gate simply fitted at its ends against planed guides in the pier faces, the contact being kept close by self-adjusting slips, and the gate was free to press down fairly on a level sill. The gate was simple. It was composed of two main girders, placed equally at each side of the center of pressure; cross vertical beams connected these girders, and carried the sheeting and the top of the gate. The static pressure was eighty-one tons. The weight of the gate was eighteen and one-half tons, and the moving load, including rollers, was roughly, twenty tons. The whole was arranged to be lifted, by a pair of strong right and left screws. These screws were placed hori-

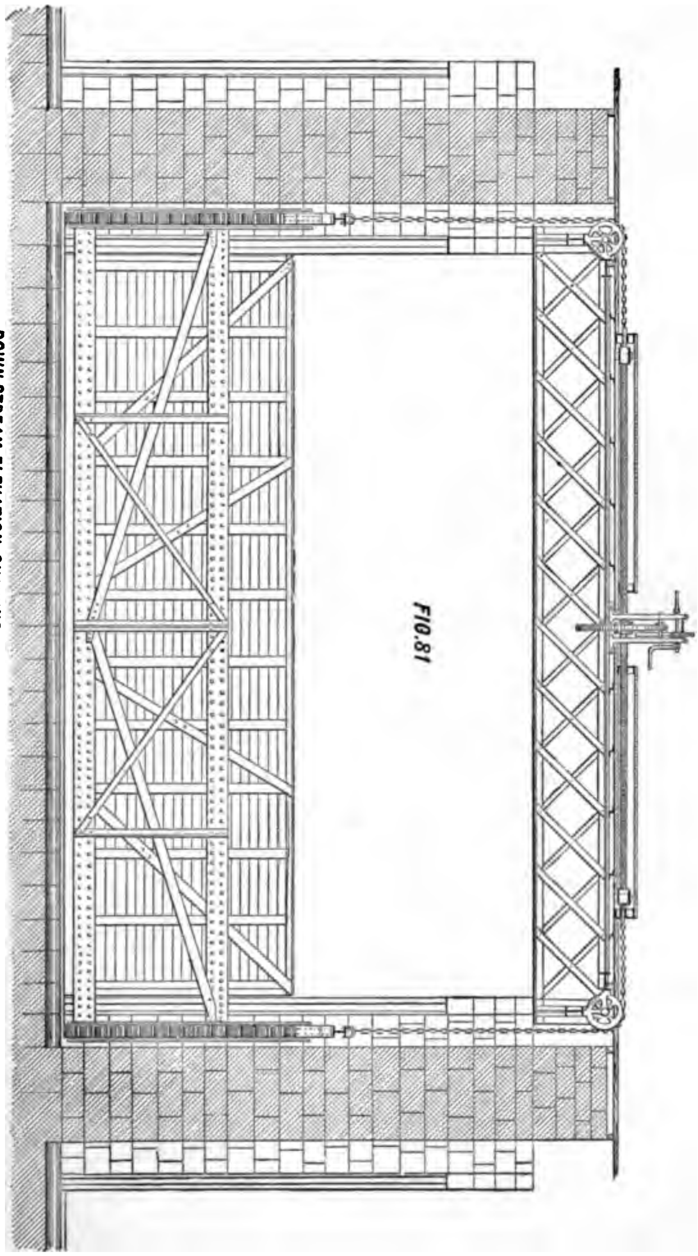
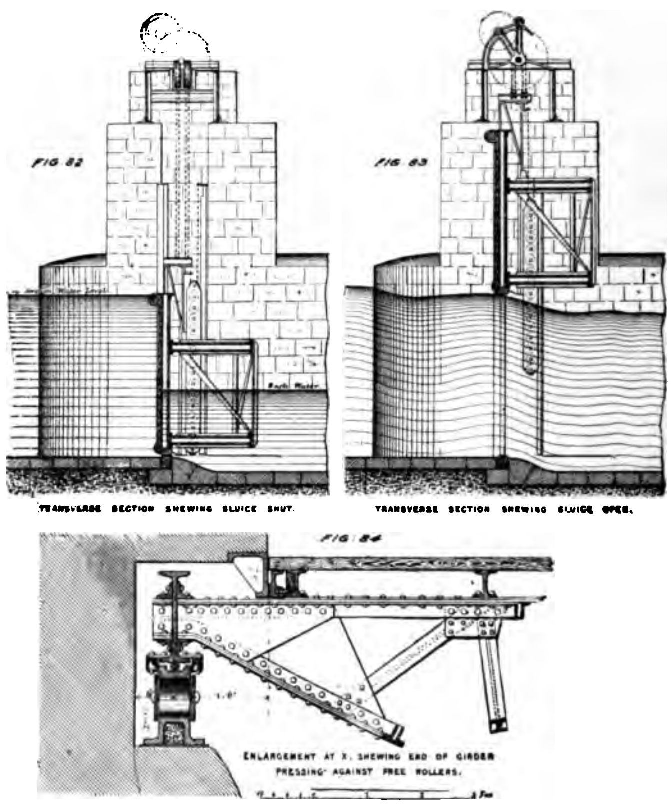


FIG. 81

Lifting Sluice Gate on Free Rollers; Clear Span 40 feet; Depth of Water 12 feet.

DOWN STREAM ELEVATION, SHEWING FOOT BRIDGE AND LIFTING GEAR.

zontally on a foot bridge, and acted on large nuts traveling in guides, each nut pulled chains which passed over large pulleys down to the gate, where they were sym-



metrically grouped round its center of gravity. The weight of this sluice-gate, foot bridge and all, was only twenty-seven tons.

Article 32. The Loss of Water by Percolation Under a Weir.

While it is not practicable, by direct experiment, to find the quantity of water lost by percolation through a sandy bed under a weir, still an approximation can be found to this loss by the method given below:—

By D'Arcy's experiments the discharge of a fourteen-inch pipe filled at the bottom with sand, to a depth of from two feet to six feet, was carefully ascertained under heads ranging from three feet to forty-six feet. Dupuit, in summarizing these and other experiments, states that the discharge is directly proportional to the head, and inversely proportional to the thickness of sand, and that for one meter head and one meter thickness of filtering material, the rate of percolation is twenty-six cubic meters per day for coarse sand, and 4.5 cubic meters for the finest sand. To check percolation, then, coarse sand is required as a matrix to give stability, and fine sand to fill up the interstices. With these materials, as Brunel found at the Thames Tunnel, the water will run through at first, but soon stop.

To give an instance of the application of D'Arcy's experiments, let us find the loss by percolation under the Godavery Anicut or Weir, Figure 44. A description of this anicut has already been given in Article 28. This anicut rests upon a bed of coarse sand of unknown depth, through which an incessant percolation takes place, the water passing under the anicut, and rising to the surface of the river again, below it. There is, at the same time, a constant draught, by the three main irrigation canals, of the greater part of the available supply entering the pool above the anicut. Yet the water only falls a foot or two below the crest, in the hot-

test weather. There is a certain area of the surface of the bed of the river, above the dam, through which leakage takes place. The leakage is greatest near the dam, and gets less as the distance from the dam increases up stream. We have no means of knowing to what distance the appreciable leakage extends. Let us assume one hundred feet as a reasonable distance, and, as the total length of the anicut is 11,866 feet, we have 1,186,600 square feet of porous, sandy surface constantly leaking under a head of twelve feet to fourteen feet. Notwithstanding this immense area of porous surface, the water level in the pool above the anicut falls but little, though the total supply in the river may be less than 3,000 cubic feet per second.*

The mean distance the water would have to traverse the sand, Figure 44, in order to pass from above to below the anicut, upon the preceding hypothesis, would be about eighty meters, and since the head of water is, say, four meters, the velocity of filtration, by Dupuit's formula, already stated, with a constant of fifteen cubic meters, that is a mean of the coarse and fine sand, would be:—

$$\frac{15\times 4}{80}=\tfrac{3}{4}\text{ meter per day}=\frac{1}{3511}\text{ feet per second.}$$

Now this velocity multiplied by the area is:—

$$\frac{1186600}{3511}=338\text{ cubic feet per second or about 11 per}$$

cent. of 3,000 cubic feet per second, which is the ordinary flow down the river in the dry season.

We thus see that the loss of water is not great, even under the most favorable conditions for percolation,

*Engineering, April 28th, 1876.

with a large area and clean sand. Under ordinary circumstances, however, the interstices of the sand, above the weir, and near the surface of the bed of the river, get filled up with fine silt, and a layer of silt is formed on the bed and banks of the river, thus materially preventing the percolation.

Article 33. Bridges—Culverts.

Only a few brief remarks will be made with reference to Bridges. There are several very good works published treating very fully on this subject.

Ordinary highway bridges are required wherever roads cross the canal, to accommodate the traffic of the country.

In America, bridges on Irrigation Canals are usually constructed of timber, but in India, Egypt and Italy, they are as a rule constructed of masonry.

Bridges are sometimes combined with and form part of Regulators and Falls. There is no difficulty about the water-way of canal bridges, as the regulators and escapes enable the high water in the canal to be always kept within the limits of its intended full supply.

The Culverts for passing the drainage of the country under the canal, are, in America, of the usual timber box-culvert type.

The culvert should be amply large to pass away the flood-water, without causing much heading up above its top. Before fixing its size, it is advisable to have a rough survey made of the area draining into the culvert, and then, assuming a heavy rainfall, the number of cubic feet of water per second reaching the culvert can be found. Having fixed the grade of the culvert, we have the grade and discharge from which the required area can be found, as explained in the *Flow of Water*.

Myers has given a formula for finding approximately the required area of the culvert. It is:—

$$a = c\sqrt{\text{Drainage area in acres.}}$$

where a = cross-sectional area of culvert in square feet, and c is a variable co-efficient having the following values:—

$c = 1.0$ for slightly rolling prairie;

$c = 1.5$ for hilly ground;

$c = 4.0$ for mountainous and rocky ground.

It may appear that this is going too much into computations to design a simple culvert, but surely the results from these are, as a rule, better than mere guess work. The computations would take but a few minutes. A defective culvert, causing a breach in a canal during the irrigation season, would do a great deal of damage.

Article 34. Aqueducts—Flumes.

Aqueducts, usually called Flumes in America, are used to carry a canal over a river or other obstruction. Before adopting an aqueduct, it is advisable to investigate whether, by altering the course of the river, the latter can be made to run clear of the former. A very instructive example of this was the diversion of the Chukkee torrent on the Baree Doab Canal, in India, for the passage of which costly works were originally designed. The Chukkee, at the time of the commencement of the canal works, had two outlets. Just above the crossing point of the canal, the main channel divided; one, the larger branch, running into the river Beas, the other into the river Ravee. This latter was embanked across at the bifurcation by bowlder dams, and spurs of the same material, protected at the extremity by masonry

revetments. By these means the whole of the water was forced to flow into the Beas, and the expense of the works for the canal crossing saved.

When, however, a canal meets a river that cannot be diverted, there are three cases under one of which it may have to be crossed:

First. When the river is on a lower level than the canal.

Second. When the river is on the same level as the canal.

Third. When the river is on a higher level than the canal.

In the first case the canal is carried over the river by an *Aqueduct* or *Flume.*

In the construction of an aqueduct there are two things to attend to of vital importance. The first is that the waterway, under the aqueduct, should be amply large, in order to pass the greatest floods with safety, and the second is that the junction of the aqueduct and the earthen embankment at its ends, should be made water-tight.

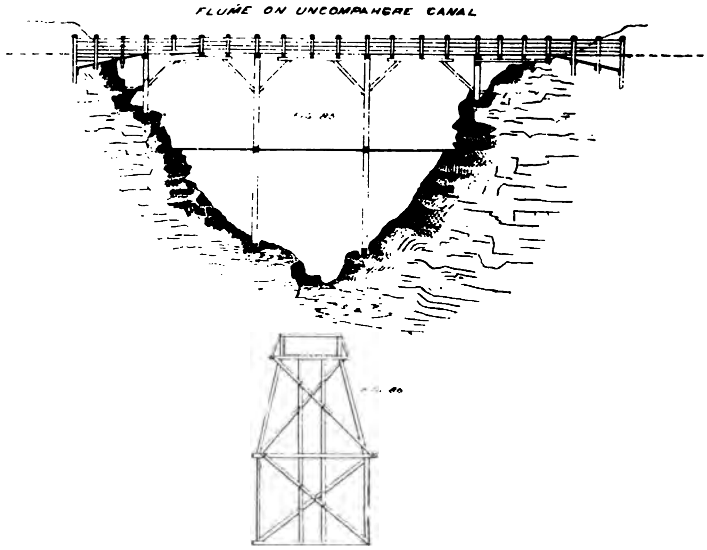
For want of ample provision for flood water the Kali Nuddee Aqueduct on the Lower Ganges Canal in India, was destroyed on January 17th, 1885, causing a loss, not only for its reconstruction, but also on account of the stoppage of irrigation.

Every practical method should be employed to find the flood discharge of the river, in order to have several checks on the results, and sufficient waterway should be provided to pass this flood water through the aqueduct, without endangering its stability in any way. For information on this subject see the articles entitled *Flow of Water.*

The embankments at the ends of the masonry aque-

duct over the river Dora Baltea, on the line of the Cavour Canal, in Italy, leaked very much at first. To remedy this the embankment was dammed up at the lower end and filled with water to a depth of about three feet. Several boats were then employed, throwing in clay all over the bed. After a time the water was turned off, and cattle driven into the muddy channel, which was turned over and worked into puddle. Again it was filled with water, and where filtration was observed, more clay was thrown in, and the puddling process repeated; and so on, till after nearly a year the filtration had entirely ceased.

FLUME ON UNCOMPAHGRE CANAL



For purposes of economy a high velocity is usually given to an aqueduct.

An aqueduct differs from a bridge in having to carry a water channel over it, instead of a road or railway, but, unlike the latter, it is not constantly subjected to the jars of a suddenly applied load.

Aqueducts are usually constructed of masonry, iron or wood, or a combination of them. The Solani Aqueduct in India, and the Dora Baltea Aqueduct in Italy, are two very fine specimens of masonry aqueducts.

A great deal of water is lost through the use of wooden troughs in flumes. When they are dry, the action of the sun causes numerous cracks which it is afterwards impossible to keep water-tight.

Figures 85 and 86 show an elevation and section of a flume on the Uncompahgre Canal in Colorado. Figures 87 and 88 show plan and section of the Big Drop on the Grand River Canal in Colorado. This drop shows another *peculiarity* of American engineering. Before reaching the drop the section of the channel is thirty feet wide by four feet deep. At the drop it descends thirty-five feet in 135 feet, and at the bottom the water is discharged against a boom of solid timbers and thrown backward in a penstock, whence it escapes over a riffled floor.

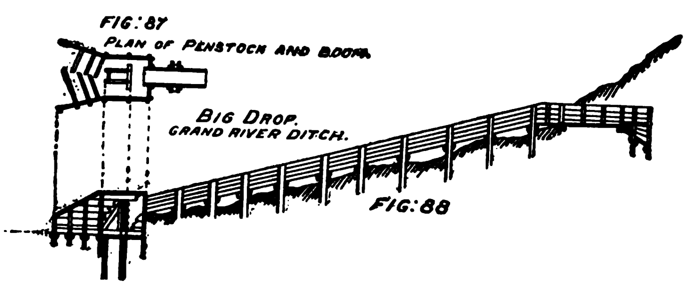


Figure 89 is a view on the Platte Canal (High Line), showing the flume issuing from tunnel, and Figure 90 is a view of flume of Platte Canal (High Line, Colorado), crossing Plum Creek at Acequia. Mr. P. O'Meara, C. E., in Transactions of the Institution of Civil Engineers, volume 73, describes flumes constructed in Colorado. The North Poudre Canal is carried from the dam

to the mouth of the cañon, through a series of tunnels and flumes, these latter supported on shelves and gulch bridges.

Platte River, with Platte Canal, Colorado.

The shelves are cut, for the most part, in the solid rock, and are nine feet wide at the base. The flume which rests on them, as on the bridges, is eight feet

Fig. 90. Aqueduct of Platte Canal (High Line) Crossing Plum Creek at Acequia.

wide by six feet deep in the clear, and projects a little over the edge, a few running beams and upright props being occasionally used to support it where fissures occur. Lower down, where the canal crosses some creeks or rivulets, the flumes are twelve feet wide by four feet three inches deep in the clear, to accord better with the section of the canal in the open plain, which is twenty feet wide at the bottom and four feet three inches deep. Precautions are taken to secure the sides of the canal for a short distance from the ends of the flume against the effects of increased speed in the water, and a slight allowance is made at these points in the general gradient.

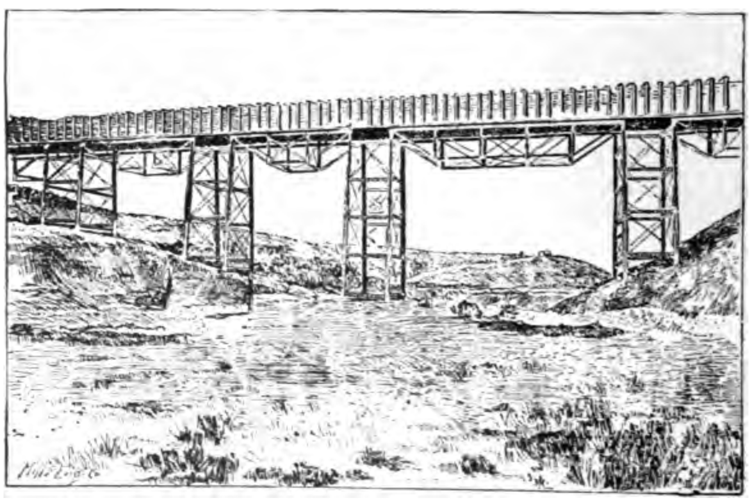


Fig. 91. High Flume over Malad River, West Branch, Utah.

The whole of the flume work is carefully calked with oakum. In earthwork the ends of the flumes are raked off to a slope one and one-half to one, and the spaces between the ties and posts, for about twelve feet back, are filled with retentive clay. Where the flume terminates in a rock cutting or tunnel, the side of the flume near the end and the rock is built up with cement masonry, for

three or four feet in length, or with two faces of masonry filled between with clay.

Mr. O'Meara further stated that the soil was very stony and pervious to water, and therefore, it was found necessary to use wooden flumes instead of embankments to carry the water. In one case about a quarter of a mile of the North Poudre Canal was embanked, and water let into it, and the consequence was that nearly all of it was carried away, and a flume of wood had to be inserted, and calked well to prevent the water from escaping.

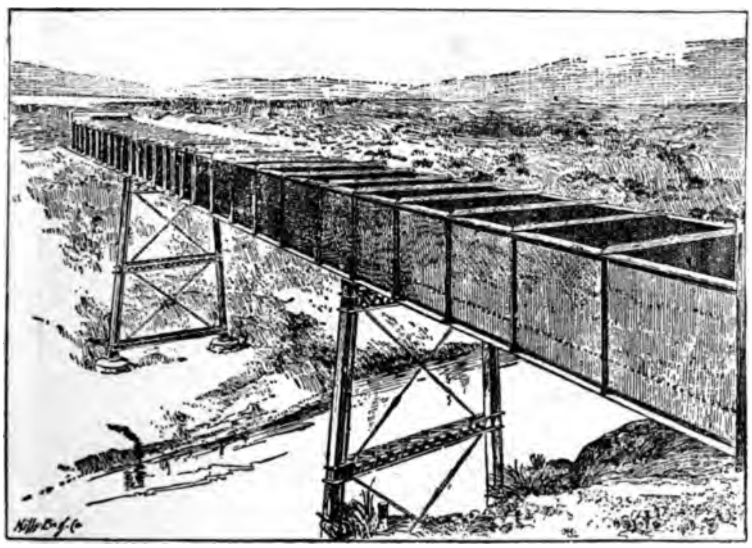


Fig. 92. Iron Flume over Malad River, Corinne Branch, Utah.

Figure 91 shows the high iron flume over the Malad river at the ninth mile of the Bear River Canal in Utah. This flume is 378 feet in length and eighty feet in maximum height, supported on iron trestles, the river span of which is seventy feet. The trough of this flume is constructed of wood. It is twenty feet wide in the clear and is intended to carry seven feet in depth of

Fig. 93. View of Solani Aqueduct, Ganges Canal.

water, and the approaches to the iron trestle are also wooden flumes of similar dimensions, and 500 feet in length.

Figure 92 shows the iron flume over the Malad river on the Corinne Branch of the Bear River Canal. This flume is founded on piles and iron cylinders filled with concrete. This flume consists of three principal bents from twenty-five to sixty feet in length, the peculiarity of its construction being that the superstructure forming the bridge itself is of iron plate girders and constitutes the flume which carries the water.*

Probably the most celebrated aqueduct in existence is the Solani Aqueduct, near Roorkee Civil Engineering College, on the line of the Ganges Canal. A brief description of this work is herewith given.

The work by which the canal is carried across the valley of the Solani river, consists of three parts.

First. The embankment of earth and brickwork, 10,713 feet in length, from the high land on the upstream side of the canal to the Solani river. This is shown in cross-section in Figure 94.

Second. The *masonry* aqueduct over the Solani river, 920 feet in length.

Third. An embankment 2,723 feet in length, similar to Figure 94.

The earthen embankment or platform is raised to an average height of sixteen and a half feet above the country, having a base of 350 feet in width, and a breadth at top of 290 feet. On this platform the banks of the canal are formed, thirty feet in width at top, and twelve feet in depth. These banks are protected from the action of the water by lines of masonry retaining walls, formed in

*The views and descriptions of the two flumes on the Bear River Canals are taken from the Irrigation Age of July 1, 1891.

steps, extending along their entire length, or for nearly two and three-quarter miles.

The river itself is crossed by a masonry aqueduct, which is not merely the largest work of the kind in India, but one of the most remarkable for its dimensions in the world. The total length of the Solani Aqueduct is 920 feet. Its clear waterway is 750 feet, in fifteen arches of fifty feet span each. The breadth of each arch is 192 feet. Its thickness is five feet; its form is that of a segment of a circle, with a rise of eight feet. The piers rest upon blocks of masonry, sunk twenty feet deep in the bed of the river, being cubes of twenty feet side, pierced with four wells each, and under-sunk in the usual manner. These foundations, throughout the whole structure, are secured by every device that knowledge or experience could suggest; and the quantity of masonry sunk beneath the surface is scarcely less than that visible above it. The piers are ten feet thick at the springing of the arches, and twelve and a half feet in height. The total height of the structure above the valley of the river is thirty-eight feet. It is not, therefore, an imposing work when viewed from below, in consequence of this deficiency of elevation; but when viewed from above, and when its immense breadth is observed, with its line of masonry channel, nearly three miles in length, the effect is most striking.

The water-way of the masonry channel is formed in two separate channels, each eighty-five feet in width; the side walls are eight feet thick and twelve feet deep, the depth of water at full supply being ten feet. A continuation of the earthen aqueduct, about three-quarters of a mile in length, connects the masonry work with the high bank at Roorkee, and brings the canal to the termination of the difficult portion of its course.

The aqueduct has carried over 7,000 cubic feet of

water per second, but usually carries between 5,000 and 6,000 cubic feet per second.

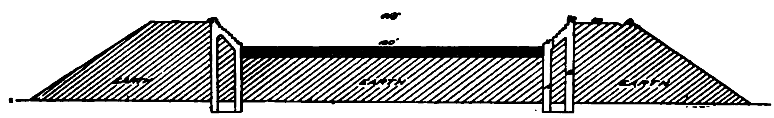
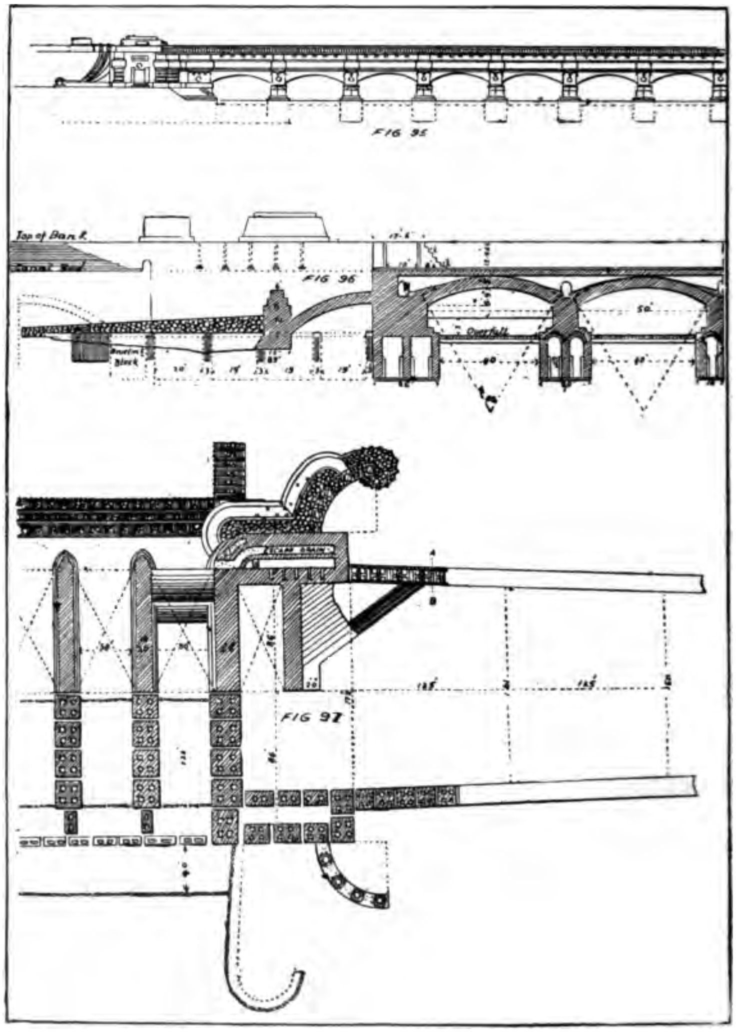


Fig. 94. Cross-Section of Solani Aqueduct Embankment.

Captain J. Crofton, R. E., in his Report on the Ganges Canal, states that: "The aqueduct over the Solani torrent has stood well. The state of the bed of the torrent above and below shows that the waterway under the aqueduct is just sufficient and no more; there is no hole or retrogression of levels down stream, and little or no silt has been deposited except under the side arches, where a certain quantity would naturally be left on the subsidence of floods. The flooring of the canal channel above requires some waterproof covering; dripping still continues through the arches, though less than at first, the effect of which has been to loosen a considerable surface of the outside plaster, and here and there bricks of an inferior description (of which it is next to impossible to prevent a few finding their way into so great a mass of brickwork) may be seen slowly decomposing from the same cause. It was at one time supposed that the pores of the brickwork would gradually fill up and so stop the percolation, but the fact is, that even the very best of brickwork is of too absorbent a nature to be proof by itself against the constant pressure of a head of water even much less than that passing over this aqueduct.

"A breach occurred some years since along a short portion of the right bank revetment, Figure 94, from the heeling over inwards, towards the channel of the wall, *A*, from a point some four or five feet below the level of the bed. The channel here had been considerably

deepened, and the accident occurred just after the canal had been laid dry. The outer wall, *B*, was very little, if at all affected.



Solani Aqueduct, Ganges Canal.

"From the investigation made immediately after its occurrence, it appears to have been caused by the pres-

sure of the earth filling, between the walls, which had become saturated by the percolation through the brickwork. When the counteracting pressure of the water in the canal was removed, the thin wall gave way, there being no weep holes through it by which the drainage from the backing could find an exit. The bed all along between these revetments was to have been protected by a layer of bowlders; this, however, has been deferred from economical motives, the actual protecting work now being confined to a sloping talus of bowlders and brick kiln rubbish, thrown down from time to time, along the foot of the revetments."

"The Henares Canal* is carried over the Majanar *Arroyo* or torrent, in an iron aqueduct which is worthy of admiration on account of its perfect fitness, both in design and construction, for the work it has to perform. The discharge of the canal at full supply is 177 feet per second."

Full details are given in Figures 98, 99, 100, 101, 102, 103.

The iron trough is seventy feet long, with a clear bearing of sixty-two feet. Its waterway is 10.17 feet wide, the sides being composed of iron box-girders 6.2 feet deep. The total weight of iron in the trough is 27.3 tons, and the weight of water when full is ninety tons. Each girder is calculated to bear 200 tons, equally distributed, or the whole trough 400 tons. The aqueduct is absolutely free from leakage, which was most ingeniously prevented. The ends of the trough rest on stone templates. Four inches from each end a pillow, composed of long strips of felt carpet, about nine inches wide, soaked in tallow, is let into the stone right across, below the breadth of the trough, which pressing fully

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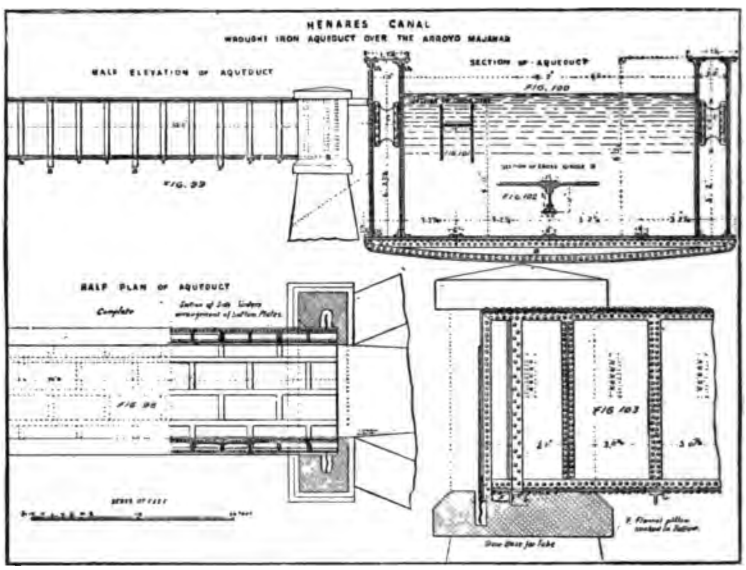
on it, makes a water-tight joint without taking the bearing off the stonework. Still further to make things secure, a recess about one foot deep and four inches wide, is cut in the stone all around the bottom and sides. In this rests a lead flushing, riveted to the trough like a



fringe. Round this lead is poured in, a hot mixture of pitch, gas tar, and fine sand, forming a water-tight joint, and yet flexible enough to allow a slight play, as required by the expansion and contraction of the iron trough. The result produced is perfect in preventing the loss of water by leakage.

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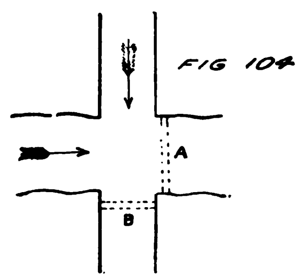
The second case, mentioned at page 151, where a canal crosses a river on the same level, is called a Level Crossing. Small drainage channels carrying small quantities of silt, may be passed into the canal without doing

any material damage. If the river, or torrent, is of large dimensions, and brings down a great volume of water at a high velocity, the above method will not answer; as



Level Crossing, Ganges Canal.

the water loaded with silt would in some places choke up the canal bed and cause the water to overflow its bank, and in other places it would erode the banks and cause breaches in them. Arrangements have, therefore, to be made to pass the flood water across the canal, which will be briefly explained, and will be understood from the following description of a level crossing shown in plan, in Figure 104:—

B is a regulating bridge across the canal, provided with the usual sluice-gates. *A* is a dam across the channel of the torrent, provided with flood-gates. Under ordinary circumstances, *A* is closed and *B* is open, so that the canal water flows along its own channel as usual. But when the torrent is in flood, then *A* must be open and *B* closed, so that the flood water may cross the canal and run down its own channel. The quantity of water flowing past the dam is likely to be, on some occasions, equal to the flood discharge of the torrent, in addition to the full supply of the canal.

The bed and banks of the canal and torrent, as far as they are exposed to the erosive action of the water, must be paved or otherwise protected, to prevent them from being injured by the action of the water.

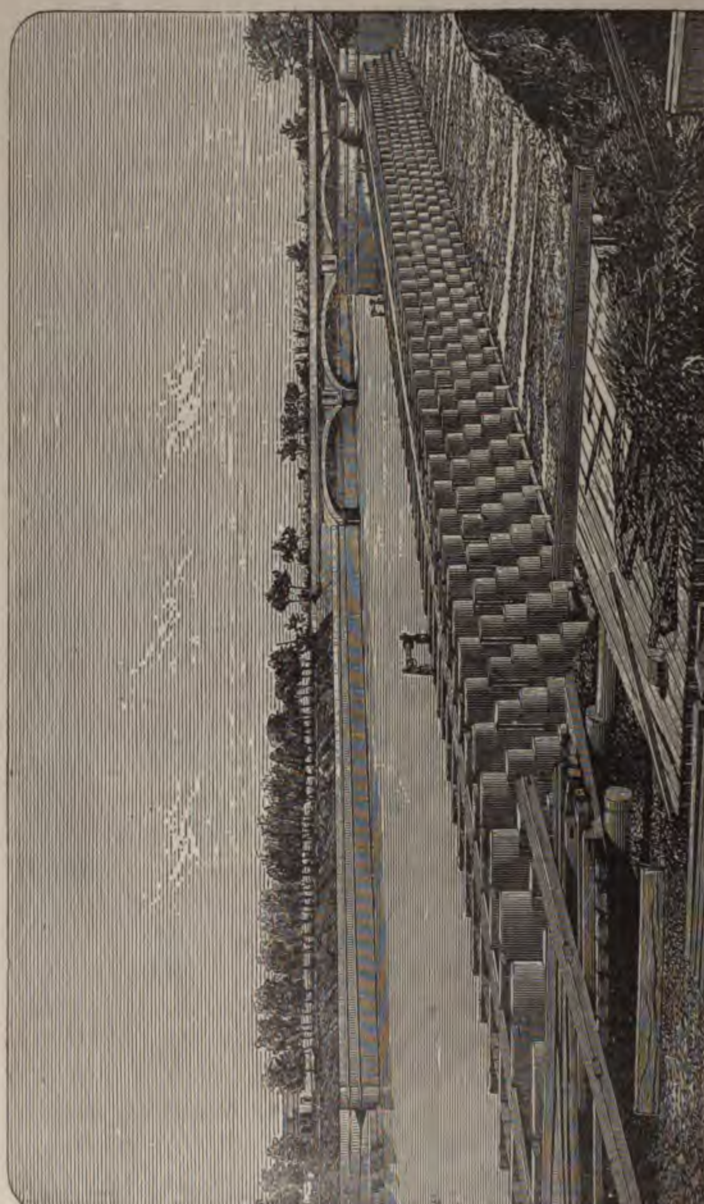


Fig. 105. View of Dhunowree Level Crossing, Ganges Canal.

A very good example of a level crossing is at Dhunowree, on the Upper Ganges Canal, where the Rutmoo torrent is passed. Figure 105 shows a view of the bridge and dam at this crossing.

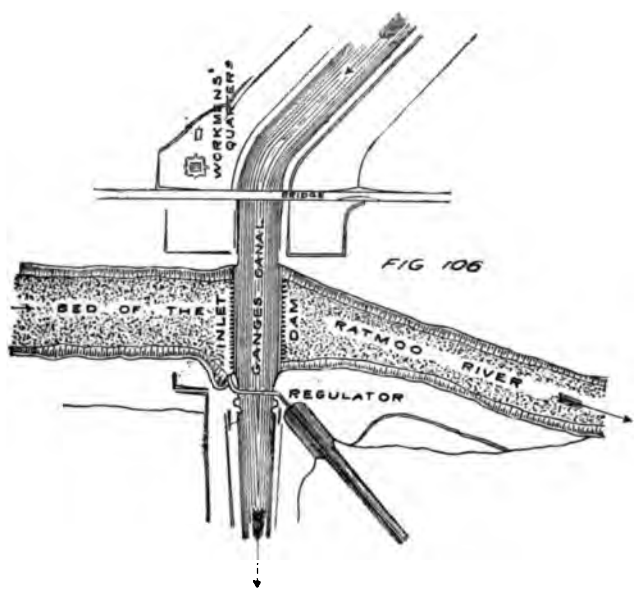


Fig. 106. Rutmoo Crossing, Ganges Canal.

Figure 106 shows the plan of the level crossing.

The dam itself consists of forty-seven sluices of ten feet in width, some of which are shown in Figure 108, with their sills flush with the canal bed, separated by piers of three and a half feet in width. The above are flanked on each side by five overfalls of the same width, having the sills raised to a height of six feet, with intermediate piers of the same dimensions as those in the center sluices. On the extreme flanks are platforms raised to a height of ten feet above the canal bed, and corresponding in height with the rest of the piers. These elevated platforms, which are seventeen feet in

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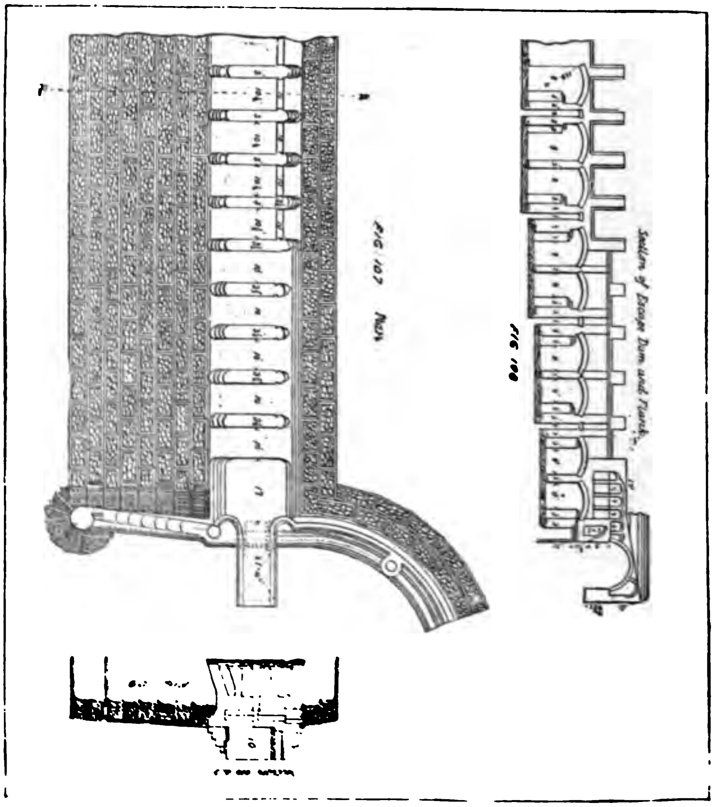

Dhunowree Level Crossing, Ganges Canal.

The amount of waterway, therefore, through the sluices, up to a height of six feet, is equal to 470 feet in width, to a height of from six to ten feet it is increased to 570 feet, and when flood water passes over the full expanse of the masonry, which is equal in width to 800 feet.

For the ten sluices in the flanks, the closing and open-

ing is effected by sleeper planks, for which grooves are fitted to the piers.

For the center openings drop gates are provided, as explained in Article 31, and Figures 67 and 68.

On the down-stream side of the dam a platform of box-work, filled with river stone, extends to a width of forty-three and a half feet from the masonry flooring. This is held in position by double lines of twenty-feet piling, strongly clamped together by sleepers fastened on to the upper surface, the slope of which is two and a quarter feet on an incline down-stream.

The regulating bridge has ten waterways each twenty feet broad, and provided with gates to prevent any flood-water passing down the canal. In addition to this, there is a roadway bridge, and about a mile of revetment walls, all resting on blocks of brick masonry, sunk to a depth of twenty feet below the canal bed. The whole of this work is protected by a forest of piles, and an enormous number of bottomless boxes filled with bowlders.

The river, when not in flood, flows under the canal by a double tunnel upwards of 500 feet long.

The great objection to this kind of work is, that it requires a permanent establishment of men on the spot to work it, and that, if they are careless or neglectful, a sudden flood may do serious damage. On this account level crossings are to be avoided whenever it is possible to do so.*

Article 36. Superpassages.

The *third* case mentioned at page 151, where the torrent crosses at a higher level than the canal, and, in this case, the structure is called a *superpassage*, to distinguish it from the first case, where the canal flows over a river and is carried by an aqueduct. In America a superpassage is usually called a flume.

*Roorkee Treatise on Civil Engineering.

Carrying a large body of water across and over a canal is a very expensive and troublesome work, as a large water channel has to be provided to carry any extraordinary flood over the canal in safety, and, in navigable canals, sufficient headway must be allowed under the superpassage so as not to interrupt the navigation. When the grade of the country admits of it, as is almost always the case, the canal can be dropped to the required level by a masonry fall, a lock being provided for navigation purposes, if required. The torrent will probably require constant watching to prevent its shifting its course and attacking the canal bank.

When not in flood a superpassage can be used as a highway bridge.

The superpassage possesses the great advantage of keeping the canal completely free from any influx of flood-water from the torrent, which is always more or less heavily charged with silt. It has the additional recommendation of not requiring the maintenance of a large establishment every rainy season, as in the case of a level crossing, where the regulating apparatus must be worked by manual labor. And lastly, the canal supply can thus be kept up without interruption, there being no necessity to shut it off at the crossing to keep the silt-laden flood-water out of the canal. The recommendations apply equally to a passage by *aqueduct*, and render them both generally preferable to a level crossing, such as that at Dhunowree, given in Article 35, when the levels will admit of the substitution.

There are two fine examples of superpassages a few miles below the headworks of the Upper Ganges Canal, by which the Puttri and Ranipore torrents are carried across the canal. These have a clear waterway between the parapets of 200 and 300 feet, respectively, and when

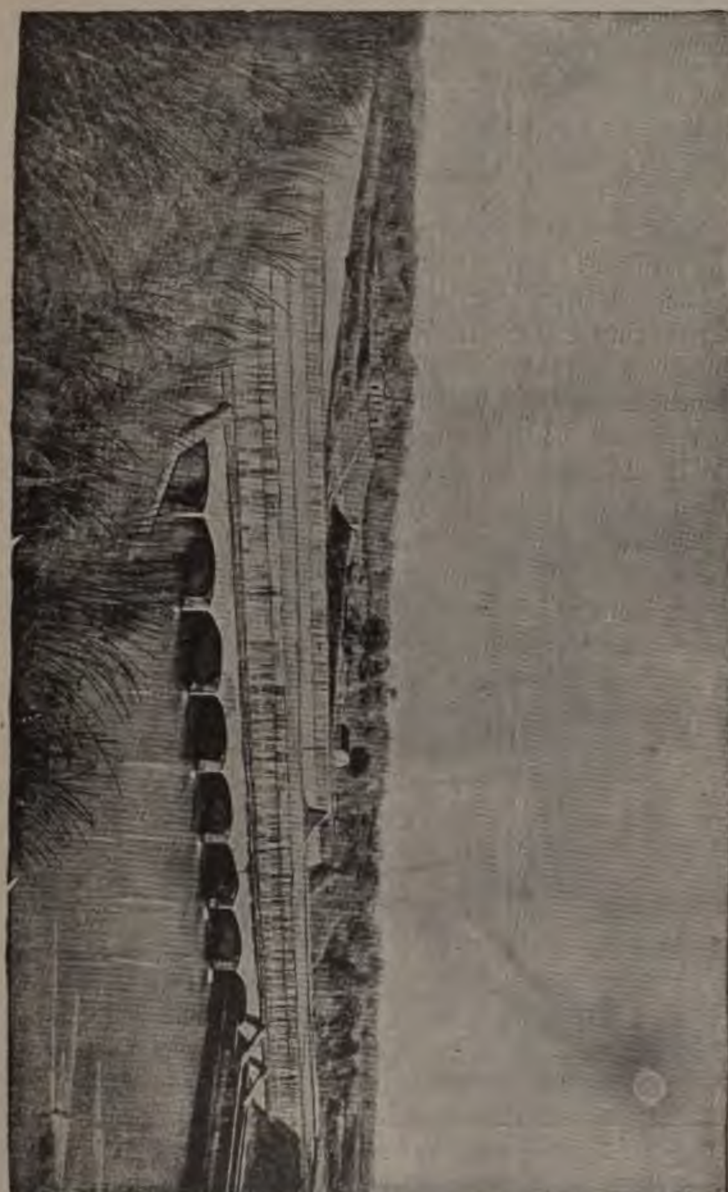

Fig. 110. RANIPORE SUPERPASSAGE, GANGES CANAL.

the torrents are not in flood, they are used as bridges of communication.*

Figure 110 gives a view of the Ranipore Superpassage. It is taken from Irrigation in India, by Mr. H. M. Wilson, M. Am. Sec., C. E., in Transactions American Society of Civil Engineers, Volume 23.

The Seesooan Superpassage on the Sutlej Canal† is shown in Figures 111, 112, 113 and 114.

Taking the catchment basin of the Seesooan to be eight miles in length by three miles in width, we obtain an area of twenty-four square miles, which would give a maximum rainfall, at the rate of half an inch per hour, of 7,752 cubic feet per second, agreeing very closely with the discharge calculated from the area of the section at the canal crossing, with the velocity due to a declivity of 1 in 791. To pass off this discharge, a water-way of 150 feet wide by six and a half feet in depth was given to the masonry channel of the superpassage. This would require a mean velocity of about eight feet per second. The dimensions given are more than ample for the required discharge, even with the worst description of masonry surface. Major Crofton, the designer of this work, computed the velocity by one of the old formulæ, having a *constant* value of c, that is:

$$v = 93\sqrt{rs}$$

Computing the mean velocity through the masonry channel, by Kutter's formula, and with different values of n, suitable to masonry surfaces, we obtain the results given in Table 16. The water channel is 150 feet wide, with vertical sides six and a half feet deep, as shown in Figure 114. The slope is 1 in 794. In round numbers,

*Roorkee Treatise on Civil Engineering.

†Report on the Sutlej Canal, by Major J. Crofton, R. E.

$\sqrt{r}$ is equal to 2.4 feet. For further information on this subject, see *Flow of Water*.

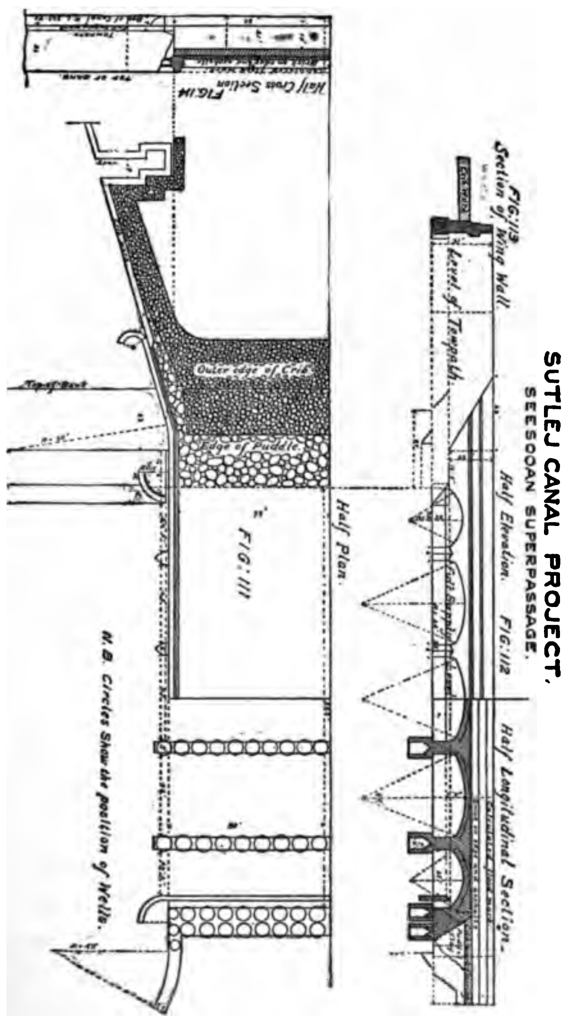

TABLE 16. Giving velocities and discharges of channels with different values of n.

Value of n	$\sqrt{r}$ in feet.	Slope 1 in 794 $\sqrt{s}$	Mean velocity in feet per second. v	Discharge in cubic feet per second. Q
.013	2.4	.035489	12.6	12,285
.015	2.4	.035489	11.0	10,725
.017	2.4	.035489	9.8	9,555
.020	2.4	.035489	8.4	8,190

The difference of level between the beds of the canal and the torrent is 21.93 feet, which is thus disposed of:

	Feet.
Depth of water in canal	7.00
Head up to soffit of arch (for navigation)	10.00
Thickness of arch	3 00
Brick-on-edge flooring	1.93
Total	21.93

The canal channel will be spanned by three central arches of forty-five feet span each, and two at the sides of thirty-two feet each; tow-paths, of seven and a half feet wide in the clear, will be carried under each side arch, leaving an aggregate water-way of 184 feet. The mean water-way of the earthen channel of the canal is only 177 feet. The addition is made to this work, in consideration of the expense of increasing its dimensions should the canal be required to carry a larger supply hereafter. The water-way for the torrent above the canal (Figure 114 shows one-half of this channel), is projected in one channel 150 feet wide at the bottom, with side walls (head walls of the work) ten feet in height, five feet thick at the base, four feet at top; the flooring over the arches to be formed of asphalt or some substance

impervious to water, the upper surface being covered with some hard material, probably a layer of kunkur (nodular limestone) slabs. The backing of the abutments will be of puddled clay, covered with a flooring of kunkur, slag, or bowlders packed in cribs.

Article 37. Inverted Syphons.

An *inverted syphon*, sometimes called a syphon, is, in some cases, used instead of an aqueduct or super-passage. The levels of the channels which cross each other determine the work best suited for the locality. The syphon is *under pressure*, and it is usual to give it such a head, or fall of water surface, from its inlet to its outlet, that it has a high velocity, and, therefore, its current has sufficient scouring force to prevent the deposition of silt and *debris*.

If the syphon has not sufficient velocity to keep itself clear, not only of the silt held in suspension, but also of the material rolled along its bed, it will in time, get partly or entirely choked, and its waters, being thus dammed, will cause floods, and very likely break the canal banks and do material damage.

Probably the most interesting inverted syphon in existence is that under Stony Creek, on the line of the Central Irrigation Canal, in Colusa County, California. It serves four purposes, namely:—

1. As an inverted syphon or conduit under Stony Creek.
2. As an escape-way for surplus canal water into the creek.
3. As a secondary gate for checking the flow of water in the canal above Stony Creek.
4. As an inlet from the creek in the lower portion of the canal.

The following description of the work is given by the designer, Mr. C. E. Grunsky, Chief Engineer of the Central Irrigation District, and the drawings, descriptive of the work, are reduced from drawings supplied by him:—

"Central Irrigation District Canal has been planned for the irrigation of 156,000 acres of land in the central portion of the west side Sacramento Valley plain.

"The canal is cut out from Sacramento River on a gradient of one in ten thousand ($6\frac{3}{8}$ inches per mile). It has a bottom width of 60 feet, and has been planned to carry a maximum depth of six feet of water. Its capacity is 730 cubic feet per second.

"In its southerly course, this canal crosses the creeks which drain the eastern slope of the Coast Range. The largest of the creeks thus crossed, is Stony Creek, which has a drainage area of 760 square miles, and a maximum flow of about 30,000 cubic feet per second.

"The grade of the bottom of the canal and the lowest point of the creek bed at the point of crossing have the same height. The width of the creek between firm banks is about 600 feet. Its bed is clean gravel. Its flow at the canal crossing generally ceases in June or July.

"The conduit under the creek, shown in Figs. 115 to 119, consists of seven semi-circular wooden tubes, constructed of long staves and covered by a common horizontal platform top. The wooden tubes are to be hung under the platform by means of iron bands, whose ends will project above longitudinal timbers on top of the platform, serving to secure the same against upward movement. The structure will be weighted with gravel and anchored to the creek bed to prevent its floating out of place.

"The conduit ends will rest on concrete in masonry inlet and outlet chambers. Each of these will be so

sure of the earth filling, between the walls, which had become saturated by the percolation through the brick-work. When the counteracting pressure of the water in the canal was removed, the thin wall gave way, there being no weep holes through it by which the drainage from the backing could find an exit. The bed all along between these revetments was to have been protected by a layer of bowlders; this, however, has been deferred from economical motives, the actual protecting work now being confined to a sloping talus of bowlders and brick kiln rubbish, thrown down from time to time, along the foot of the revetments."

"The Henares Canal* is carried over the Majanar *Arroyo* or torrent, in an iron aqueduct which is worthy of admiration on account of its perfect fitness, both in design and construction, for the work it has to perform. The discharge of the canal at full supply is 177 feet per second."

Full details are given in Figures 98, 99, 100, 101, 102, 103.

The iron trough is seventy feet long, with a clear bearing of sixty-two feet. Its waterway is 10.17 feet wide, the sides being composed of iron box-girders 6.2 feet deep. The total weight of iron in the trough is 27.3 tons, and the weight of water when full is ninety tons. Each girder is calculated to bear 200 tons, equally distributed, or the whole trough 400 tons. The aqueduct is absolutely free from leakage, which was most ingeniously prevented. The ends of the trough rest on stone templates. Four inches from each end a pillow, composed of long strips of felt carpet, about nine inches wide, soaked in tallow, is let into the stone right across, below the breadth of the trough, which pressing fully

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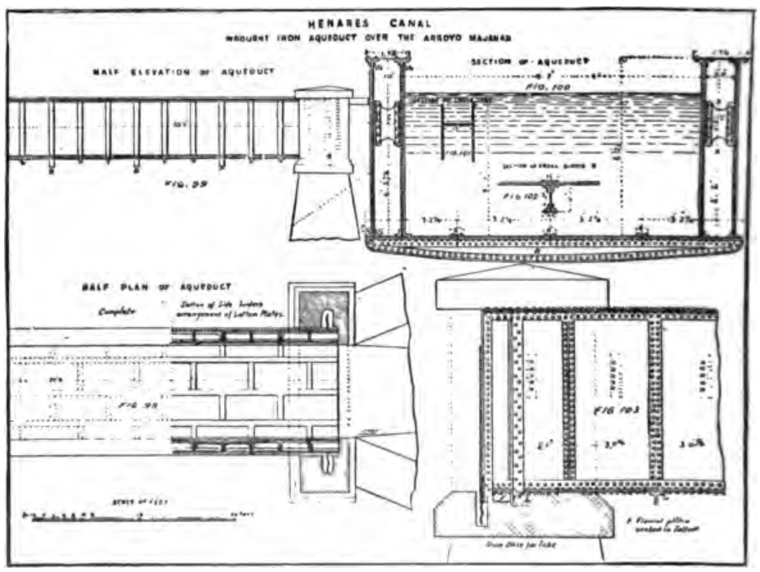

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The second case, mentioned at page 151, where a canal crosses a river on the same level, is called a Level Crossing. Small drainage channels carrying small quantities of silt, may be passed into the canal without doing

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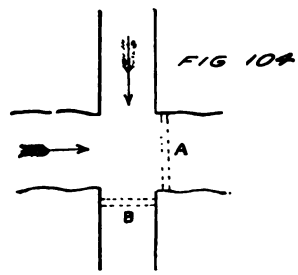

Level Crossing, Ganges Canal.

the water loaded with silt would in some places choke up the canal bed and cause the water to overflow its bank, and in other places it would erode the banks and cause breaches in them. Arrangements have, therefore, to be made to pass the flood water across the canal, which will be briefly explained, and will be understood from the following description of a level crossing shown in plan, in Figure 104:—

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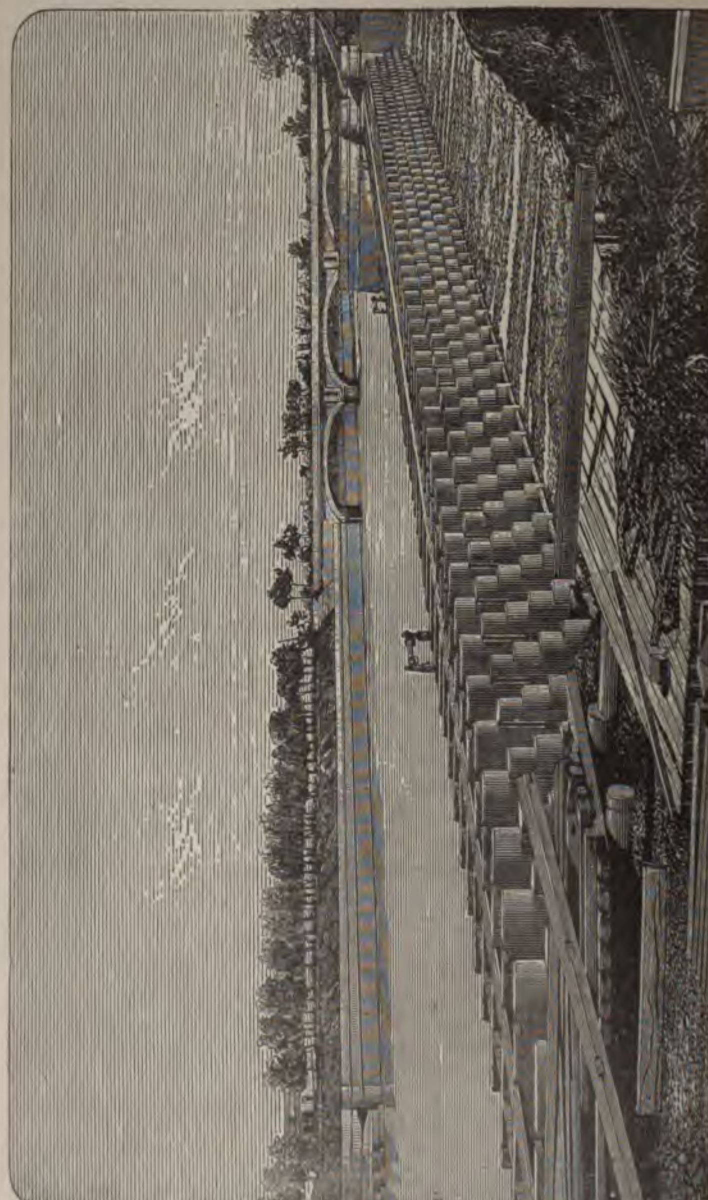

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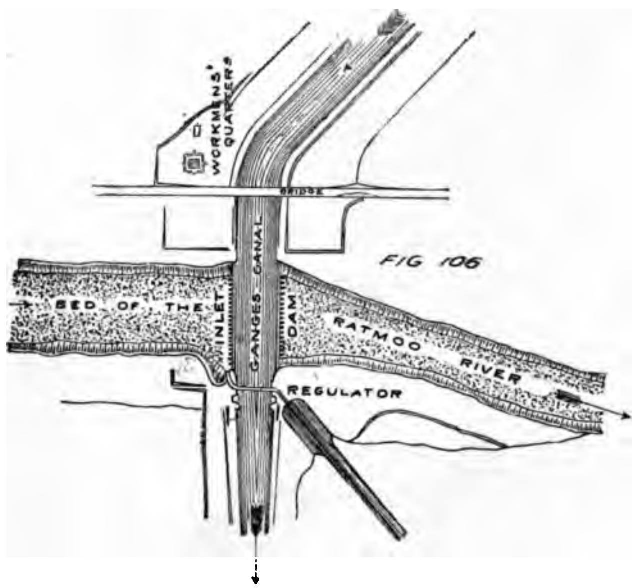

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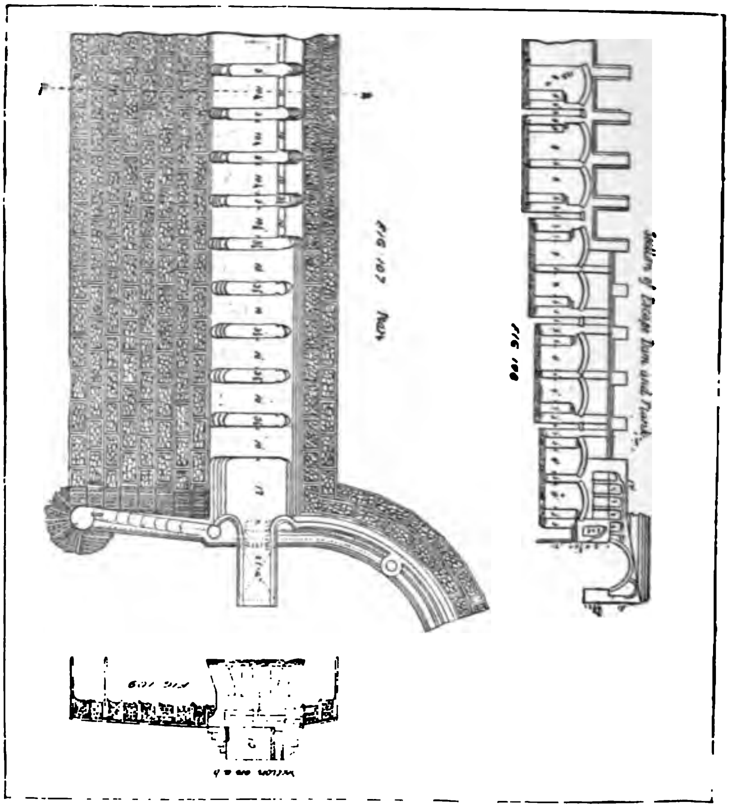

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The great objection to this kind of work is, that it requires a permanent establishment of men on the spot to work it, and that, if they are careless or neglectful, a sudden flood may do serious damage. On this account level crossings are to be avoided whenever it is possible to do so.*

Article 36. Superpassages.

The *third* case mentioned at page 151, where the torrent crosses at a higher level than the canal, and, in this case, the structure is called a *superpassage*, to distinguish it from the first case, where the canal flows over a river and is carried by an aqueduct. In America a superpassage is usually called a flume.

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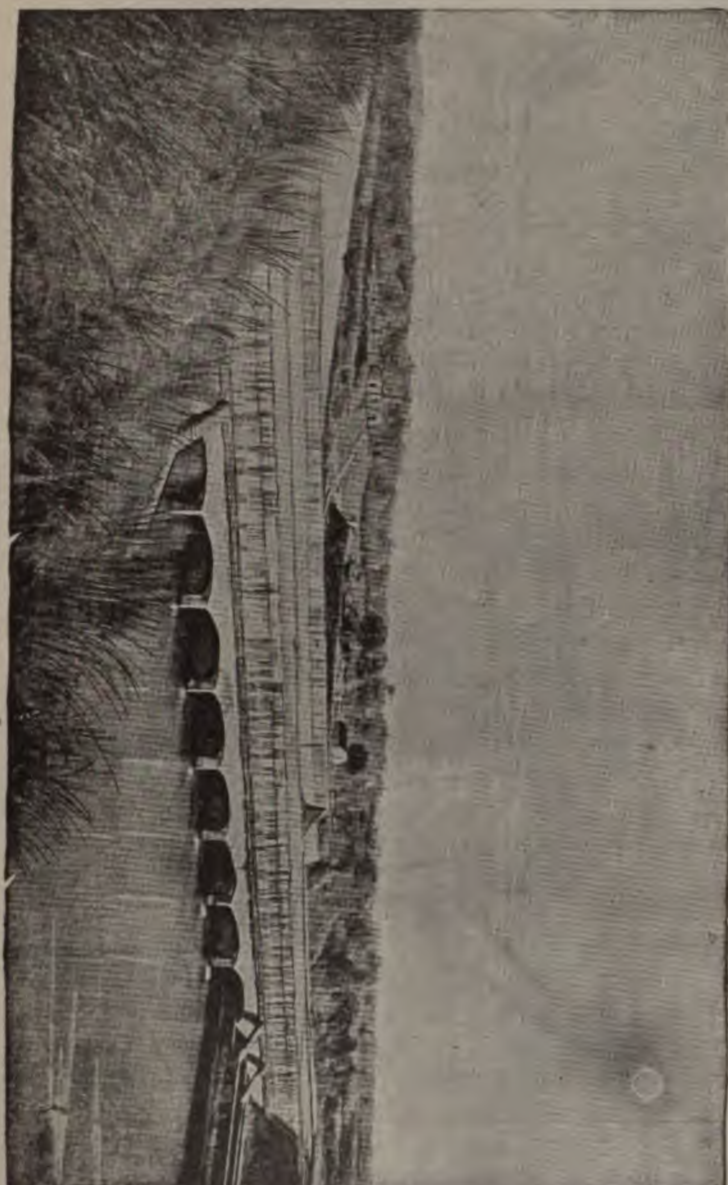

Fig. 110. RANIPORE SUPERPASSAGE, GANGES CANAL.

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The Seesooan Superpassage on the Sutlej Canal† is shown in Figures 111, 112, 113 and 114.

Taking the catchment basin of the Seesooan to be eight miles in length by three miles in width, we obtain an area of twenty-four square miles, which would give a maximum rainfall, at the rate of half an inch per hour, of 7,752 cubic feet per second, agreeing very closely with the discharge calculated from the area of the section at the canal crossing, with the velocity due to a declivity of 1 in 791. To pass off this discharge, a water-way of 150 feet wide by six and a half feet in depth was given to the masonry channel of the superpassage. This would require a mean velocity of about eight feet per second. The dimensions given are more than ample for the required discharge, even with the worst description of masonry surface. Major Crofton, the designer of this work, computed the velocity by one of the old formulæ, having a *constant* value of c, that is:

$$v = 93\sqrt{rs}$$

Computing the mean velocity through the masonry channel, by Kutter's formula, and with different values of n, suitable to masonry surfaces, we obtain the results given in Table 16. The water channel is 150 feet wide, with vertical sides six and a half feet deep, as shown in Figure 114. The slope is 1 in 794. In round numbers,

*Roorkee Treatise on Civil Engineering.

†Report on the Sutlej Canal, by Major J. Crofton, R. E.

$\sqrt{r}$ is equal to 2.4 feet. For further information on this subject, see *Flow of Water*.

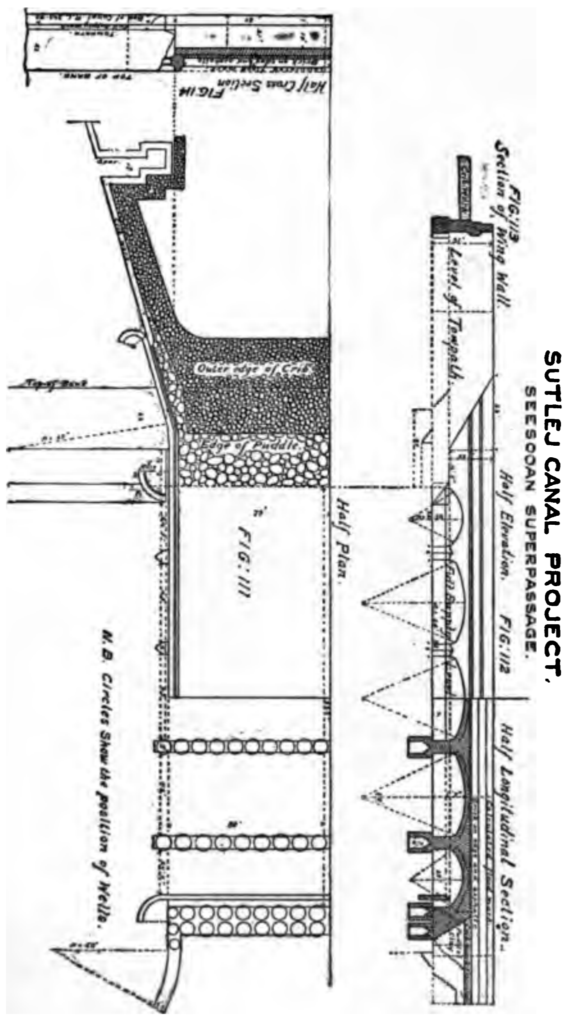


TABLE 16. Giving velocities and discharges of channels with different values of n.

Value of n	$\sqrt{r}$ in feet.	Slope 1 in 794 $\sqrt{s}$	Mean velocity in feet per second. v	Discharge in cubic feet per second. Q
.013	2.4	.035489	12.6	12,285
.015	2.4	.035489	11.0	10,725
.017	2.4	.035489	9.8	9,555
.020	2.4	.035489	8.4	8,190

The difference of level between the beds of the canal and the torrent is 21.93 feet, which is thus disposed of:

	Feet.
Depth of water in canal	7.00
Head up to soffit of arch (for navigation)	10.00
Thickness of arch	3 00
Brick-on-edge flooring	1.93
Total	21.93

The canal channel will be spanned by three central arches of forty-five feet span each, and two at the sides of thirty-two feet each; tow-paths, of seven and a half feet wide in the clear, will be carried under each side arch, leaving an aggregate water-way of 184 feet. The mean water-way of the earthen channel of the canal is only 177 feet. The addition is made to this work, in consideration of the expense of increasing its dimensions should the canal be required to carry a larger supply hereafter. The water-way for the torrent above the canal (Figure 114 shows one-half of this channel), is projected in one channel 150 feet wide at the bottom, with side walls (head walls of the work) ten feet in height, five feet thick at the base, four feet at top; the flooring over the arches to be formed of asphalt or some substance

impervious to water, the upper surface being covered with some hard material, probably a layer of kunkur (nodular limestone) slabs. The backing of the abutments will be of puddled clay, covered with a flooring of kunkur, slag, or bowlders packed in cribs.

Article 37. Inverted Syphons.

An *inverted syphon*, sometimes called a syphon, is, in some cases, used instead of an aqueduct or super-passage. The levels of the channels which cross each other determine the work best suited for the locality. The syphon is *under pressure*, and it is usual to give it such a head, or fall of water surface, from its inlet to its outlet, that it has a high velocity, and, therefore, its current has sufficient scouring force to prevent the deposition of silt and *debris*.

If the syphon has not sufficient velocity to keep itself clear, not only of the silt held in suspension, but also of the material rolled along its bed, it will in time, get partly or entirely choked, and its waters, being thus dammed, will cause floods, and very likely break the canal banks and do material damage.

Probably the most interesting inverted syphon in existence is that under Stony Creek, on the line of the Central Irrigation Canal, in Colusa County, California. It serves four purposes, namely:—

1. As an inverted syphon or conduit under Stony Creek.
2. As an escape-way for surplus canal water into the creek.
3. As a secondary gate for checking the flow of water in the canal above Stony Creek.
4. As an inlet from the creek in the lower portion of the canal.

The following description of the work is given by the designer, Mr. C. E. Grunsky, Chief Engineer of the Central Irrigation District, and the drawings, descriptive of the work, are reduced from drawings supplied by him:—

"Central Irrigation District Canal has been planned for the irrigation of 156,000 acres of land in the central portion of the west side Sacramento Valley plain.

"The canal is cut out from Sacramento River on a gradient of one in ten thousand (6⅜ inches per mile). It has a bottom width of 60 feet, and has been planned to carry a maximum depth of six feet of water. Its capacity is 730 cubic feet per second.

"In its southerly course, this canal crosses the creeks which drain the eastern slope of the Coast Range. The largest of the creeks thus crossed, is Stony Creek, which has a drainage area of 760 square miles, and a maximum flow of about 30,000 cubic feet per second.

"The grade of the bottom of the canal and the lowest point of the creek bed at the point of crossing have the same height. The width of the creek between firm banks is about 600 feet. Its bed is clean gravel. Its flow at the canal crossing generally ceases in June or July.

"The conduit under the creek, shown in Figs. 115 to 119, consists of seven semi-circular wooden tubes, constructed of long staves and covered by a common horizontal platform top. The wooden tubes are to be hung under the platform by means of iron bands, whose ends will project above longitudinal timbers on top of the platform, serving to secure the same against upward movement. The structure will be weighted with gravel and anchored to the creek bed to prevent its floating out of place.

"The conduit ends will rest on concrete in masonry inlet and outlet chambers. Each of these will be so

constructed that the space between masonry walls can be opened or closed to the creek. The gates to accomplish this will be arranged on the flash board principle, *i. e.*, grooves along vertical posts will be provided to receive and support the ends of loose horizontal boards.

"The structure thus becomes an escape-way for canal water to Stony Creek.

"It serves as a check-weir to the main canal and can be used to regulate the volume of flow in the canal.

"It serves as an inlet for waters of Stony Creek, thus enabling the creek to be used as a secondary source of supply."

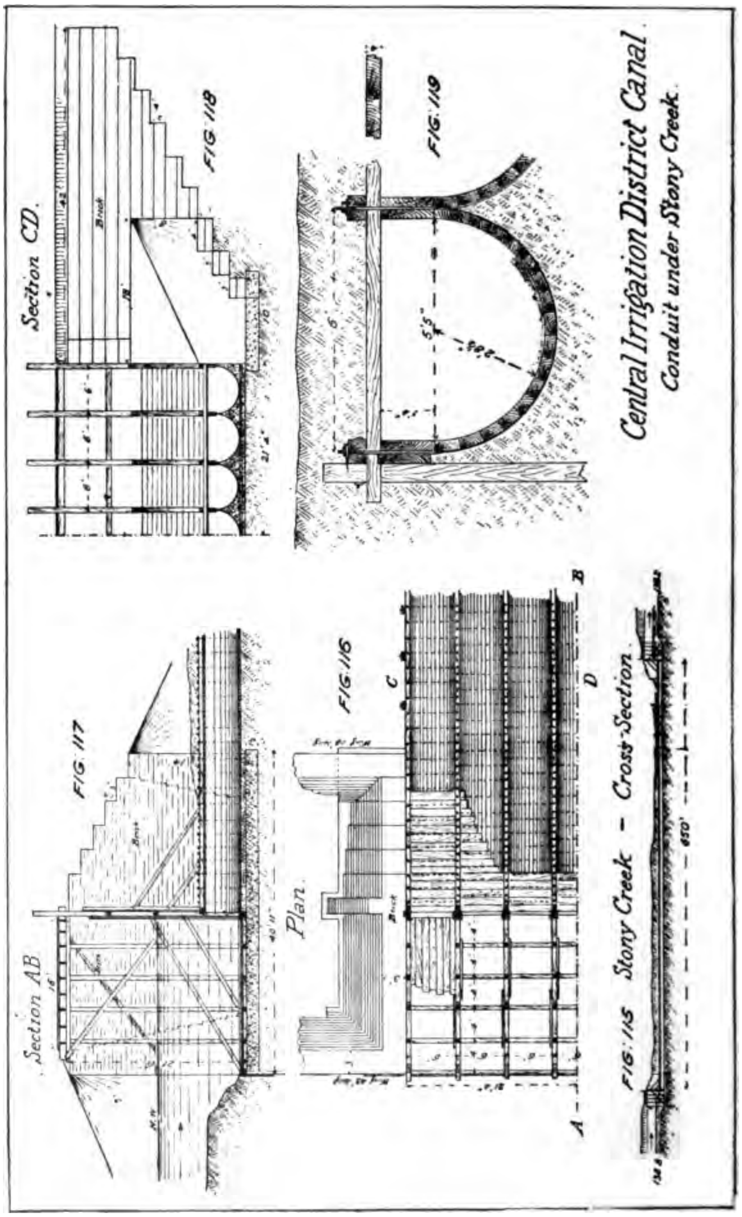
Figures 115, 116, 117, 118 and 119 show part plan and sections of this work.

The line of the Agra Canal, India, crosses the Buriya Torrent. The volume of the latter is 2,000 cubic feet per second, and it flows over a steep and rocky channel. Its passage *under* the canal is provided for by a partial syphon having seven culverts, 6 feet wide and 4 feet deep, the velocity of which, in high floods, will be 12 feet per second. This velocity is sufficient to move ordinary sized bowlders, and, therefore, sufficient to keep the channel clear of any deposit that can reach it.

The culverts are covered by large stones bolted down to the piers, and, for this purpose, bolts are built into the latter. A strong breast wall on each side supports the canal banks, and the ordinary earthen section of the canal is carried over the syphon culverts. The canal flows over the syphon without any change in its wetted cross-section or in its velocity. The syphon is provided with a floor of massive rough ashlar, the entrance and egress for the torrent being also built with large stone.

Figures 120, 121, 122 and 123 show the syphon carrying the Hurron Creek (Nullah) under the Sirhind or Sutlej Canal. This work is constructed of brick masonry.

12



Central Irrigation District Canal
Conduit under Stony Creek.

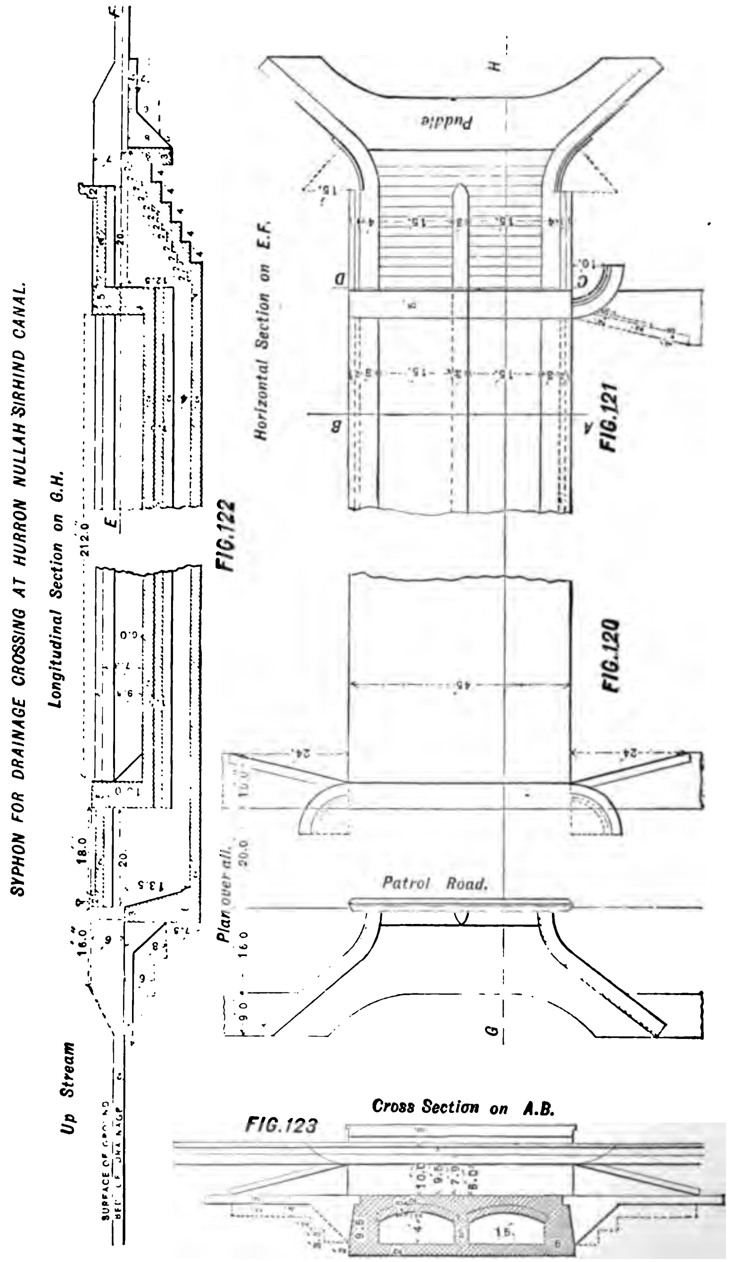
The Inverted Syphon carrying the Cavour Canal, Italy, under the Sesia torrent, is one of the finest works of the kind in existence. During extreme floods the Sesia carries about 160,000 cubic feet per second.

The syphon is 863 feet in length. It consists of five elliptically shaped conduits, or culverts, the horizontal diameter at the entrance of each being 16 feet 5 inches, and the vertical diameter of 7 feet 10 inches at the entrance, and 7 feet 6 inches at the exit. The area of the five culverts at the entrance is about 483 square feet, and the canal was designed to carry 3,883 cubic feet per second. This would give a mean velocity, at the entrance, of about 8 feet per second, and with a supply of 3,000 feet per second, it would give a mean velocity of about 6 feet per second, and, with these velocities, it is found that no silting of the culverts takes place.

The arch is of brickwork 1 foot 9 inches in thickness. On this is laid a thin layer of concrete and, again on this, planking, the upper surface of which coincides with the bed of the river.

The planks are laid in the direction of the general flow of the current of the river. The total thickness of brickwork, concrete and planking from the soffit of the arch to the bed of the torrent is only 3 feet 2 inches. The surface of the water in the canal, after passing through the syphon, is 2 feet 3 inches above the bed of the Sesia.

On the Verdon Canal in France, there are several wrought iron syphons; four of which are across deep valleys. The most important is that at St. Paul, 890 feet long, constructed of two parallel wrought-iron tubes, each 5 feet 9 inches in internal diameter, with a maximum pressure equal to $116\frac{3}{8}$ feet head of water. The capacity of this canal is equal to 212 cubic feet per second, so that at full supply the mean velocity through this syphon is equal to 8 feet per second.


SYPHON FOR DRAINAGE CROSSING AT HURRON NULLAH SIRHIND CANAL.
Longitudinal Section on G.H.
FIG.122
Horizontal Section on E.F.
FIG.121
FIG.120
Plan over all.
Patrol Road.
Puddle
Up Stream
Cross Section on A.B.
FIG.123

The horizontal portion of the syphon, laid at the bottom of the valley, is 321.6 feet in length. The remainder of its length, consisting of the two inclines, is laid at a slope of about 2½ to 1. The pipes are of wrought-iron, and respectively 0.353 inch and 0.315 inch thick for the horizontal and inclined portions. They are supported on, and fixed in, masonry at the junctions of the horizontal and inclined portions. The remainder of the lengths bear on cast-iron rollers, resting on stone blocks, placed about 31 feet apart. The arrangements for the expansion and contraction consist in constructing a short length of the tubes, in each of the horizontal and inclined portions, of a gradually increasing diameter from, and then increasing back to, the normal diameter of the tube. The metal of the tubes at these swellings is reduced to about ¼ inch in thickness; in order to obtain greater elasticity, and it is contended that the bulging in and drawing out of the tube, at these swellings, will respectively allow for expansion and contraction of the metals.

At the beginning of the works on this canal, the long syphons, including that of St. Paul, above described, were constructed through the natural rock, and lined with masonry to prevent leakage, at a depth of 50 to 82 feet below the bottom of the valleys, the rock being intended to resist the hydraulic pressure. After completion, not one of these tunnel syphons acted satisfactorily when the water was let into them. Repairs were then commenced, experimenting at first with those under the least, and ending with those under the greatest head of pressure. After much trouble and expense all were at length caused to act satisfactorily, with the exception of that of St. Paul, which had to be abandoned, and wrought iron pipes substituted as above described.

On one of the branches of this canal, there is a syphon 443 feet long, under a head of pressure equal to 71½ feet.

The Lozoya Canal, in Spain, with a discharge of 89 cubic feet per second, has six syphons, one of which consists of four cast iron pipes, each 2.8 feet in diameter, three of which carry the canal, the fourth being used in case of accidents to any of the other three.

There are two most interesting syphons on the Jucar Canal, in Spain, under the *barrancos* or torrents of Carlet and Alginet, the former 455 and the latter 524 feet long. The Carlet syphon, situated about 14½ miles below Antella, is a very old construction. Its discharge is 350 cubic feet per second. The canal is 19 feet wide just above, but diminishes to 7.5 at the entrance of the syphon, which is barred, first by an iron grating, and again by a wooden one, the bars being about six inches apart. Directly over the entrance stands the guard's house. There are two masonry shafts built in the bed of the torrent down to the syphon. They, too, are protected by gratings. The mouth is closed in masonry revetments, supported by arches, and the water-section just below is 6.75 feet wide and 8 feet deep. The fall through the syphon is 4.9 feet. M. Aymard questioned some workmen who had once been in it, and they told him the gallery was 5.9 feet wide and 6.5 feet high. He hence calculated the velocity through it to be 10 feet per second.

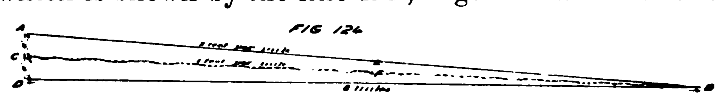
General Scott Moncrieff* says that the Moors of Spain have left many proofs of their skill in making tunnels and syphons on their irrigation canals. The Mijares Canal, which irrigates 9,800 acres of land, and close to its *prise*, or headworks, enters a tunnel 1,300 feet long; after which it is carried by a syphon 327 feet in length under the Viuda ravine. The lowest point of this

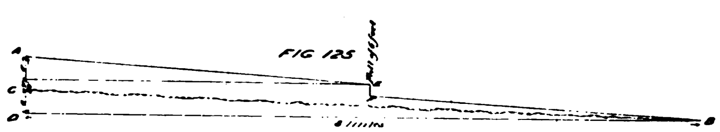
* Irrigation in Southern Europe.

syphon is about 180 feet below the mouth, and 90 feet below the present bed of the torrent which crosses it. There is a fall through it of 13 feet.

Article 38. Retrogression of Levels.

Retrogression of levels, or the lowering of the bed of a channel by erosion, *on long lines*, is a serious evil in any irrigation channel, especially in a canal carrying a large volume of water, say over 1,000 cubic feet per second. Scour or erosion is usually referred to local action of the water, while Retrogression of Levels is usually applied to long reaches of a channel. It is caused by giving too great a slope, and, therefore, too great a velocity to the canal. For example, a canal is constructed with a slope of two feet per mile, the bed of which is shown by the line $A\ B$, Figure 124. The canal





Retrogression of Levels in Canals.

has a high mean velocity, over three feet per second. The material through which the canal runs is sandy loam, which cannot bear a higher mean velocity than 2.25 feet per second without erosion. In the course of a few years the current in the canal scours the bed to such an extent that retrogression of bed levels has taken place from $A\ B$, with a grade of two feet per mile, and a mean velocity of over three feet per second, to $C\ B$, with a grade of one foot per mile, and a velocity of about 2.25 feet per second. Retrogression of levels ceases at the line

$C B$, and on this grade line the canal has established its regimen.

In another case, through the same material, the bed of the canal, from A to E, Figure 125, was given a fall of two feet per mile. At E was a vertical drop of four feet to F, and then a grade of two feet per mile from F to B. The fall $E F$ was so badly constructed that it was washed away and not rebuilt, thus practically adding four feet to the fall of the bed from A to B. Retrogression of levels took place until all scour stopped at the line $C B$ at a ruling grade of one foot per mile.

Bad results generally follow the lowering of the bed, of which a few are here mentioned.

The beds of the distributing channels taken from the main channel between A and B were fixed with reference to the bed of the canal, $A B$, Figure 124, and any lowering of this bed would diminish the depth of water at, and, therefore, the supply entering their heads. For example, midway between A and B, at E, a small channel is taken out with its bed three feet above the bed of the main channel, but after some time, retrogression of levels has lowered the bed of the main canal four feet from E to F. The surface of water in the main channel, at full supply of six feet in depth, is, therefore, one foot below the bed of the distributing channel at E. In order, therefore, to get water through the distributary, a regulating gate would have to be constructed on the main channel below E, so that, by closing this gate as required, the level of the water could be raised. Another plan would be to deepen the distributing channel for some distance from its head. This deepening of the distributing channel would flatten its grade and cause a diminution of its velocity and discharge. Furthermore, the land on each side of the distributary, for some distance from its head, would be too high above it for irrigation by gravity.

Another evil caused by lowering the canal bed would be the lowering of the sub-surface water in the land on each side of the canal, where in some cases such lowering is not required.

Any one who has inspected the old Irrigation Channels in parts of California will have seen that retrogression of levels has taken place with bad results in numerous cases. As a rule, too much grade has been given to the old canals.

In the early days of the construction of large irrigation canals, by the British Government, in India, this mistake was made. The greatest canal engineer that ever existed, General Sir Proby Cautley, the designer and constructor of that magnificent work, the original or Upper Ganges Canal, decided, after careful thought and due regard to the experience gained on canals previously opened, that 15 inches per mile was required as the grade of this canal. This slope was too great, and probably six inches per mile would have been ample. After the canal was in use for some years, retrogression of levels took place to an alarming extent; deep holes were scoured out below many of the falls and bridges, thereby endangering their stability, and the bed of the canal was in some places deepened, and in others widened beyond the original cross-section.

When Cautley fixed the slope of the Ganges Canal, nothing was then known of the experiments and investigations of Humphreys and Abbot, D'Arcy, Bazin, Gordon, Kutter and Ganguillet and others. Cautley fixed his slope in accordance with the formula of Dubuat, a formula that was used for many years in India, both before and after the opening of the Ganges Canal. Neville's Hydraulics was then a standard work, in use by the canal engineers of Northern India, and in the last edition of this work, 1875, a table based on Dubuat's

formula is given *for finding the Mean Velocity of Water flowing in Pipes, Drains, Streams and Rivers.*

No blame whatever can be attached to Cautley for determining the slope of the Ganges Canal by this formula, for it was generally used in India at the time, and was believed to be correct. It is now known to be very inaccurate.

This mistake of the slope was the only radical defect in the design of the Ganges Canal; and, with this exception, there has never been constructed a work of equal magnitude, that showed so few mistakes, or that displayed more originality and boldness in design and execution.

Notwithstanding all this, a set of carping critics made the last years of the life of General Cautley miserable, by direct and indirect, unjust attacks on his great work.

In 1864 a committee of five Royal Engineers—not a single Civil Engineer was on it—recommended that Captain J. Crofton, another Royal Engineer, should report on the remodelling of the Ganges Canal. Captain Crofton did so report in 1866, and he estimated the cost of remodelling at $12,500,000.

One able Civil Engineer, Mr. Thomas Login, proved conclusively that protective works, at a comparatively small expense, would ensure the safety of the Canal. He raised the crests of the falls in some places by planking, and crib-work filled with bowlders was placed below them in the bed of the canal. By these means water-cushions were formed below the falls, which materially reduced the destructive effect of the falling water. For a detailed description of this work see the account given by Mr. Thomas Login, C. E.* His only thanks was removal to another work, and he was never again appointed

*Transactions of the Institution of Civil Engineers. Vol. XXVII. 1867-68.

to the Ganges Canal. Mr. Login and the writer of this work were intimate friends, and in justice to the former this brief reference is made to his work on the Ganges Canal, for which he did not get the credit that he deserved.

In 1868 Mr. Login stated:* "Six years ago the Author stood almost alone in maintaining the opinion that the Ganges Canal needed only some protective work, and did not require the radical alterations then proposed. The Author will only add the expression of his firm conviction, that the works may be placed out of danger by the judicious use of wood and iron, at a less cost than if stone be employed, without depriving the country of the benefits of irrigation beyond a short time."

Time has fully justified Mr. Login's opinion, and a few years since General C. E. Moncrieff, R. E., testified to his sound judgment.† He states:

"Did they not remember how a committee of engineers had pronounced Cautley's great Ganges Canal unsound, and how they would have spent fabulous sums on it, had not Brownlow's common sense and attention to details shown that the work was all right, as it has triumphantly proved to be."

Had General Moncrieff given the credit of this success to Login instead of Brownlow, his statement would be more in accordance with the facts. It was mainly due to Login's sound judgment, sturdy independence, and having practically proved what could be done at comparatively small expense, that overtaxed India was saved from the great expense that was proposed to be incurred in remodelling the canal, and another serious matter, the closing of the canal for one or two years.

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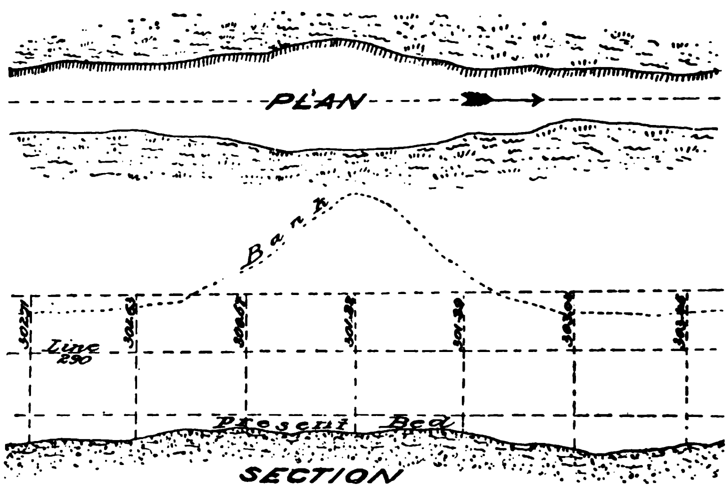
* Transactions of the Institution of Civil Engineers, Vol. XXVII, 1867-68.

† Irrigation in Egypt in Nineteenth Century. February, 1885.

Figures 126 and 127 are plan and profile of the original Ganges Canal, through the Toghulpoor sand hill in mile 37.* There was here more erosion in the width than in the depth. The plan, Figure 126, shows the widening, and the profile, Figure 127, shows that the bed of the canal through this wide section is higher than the narrow channel above and below that place.



Figs. 126, 127. Widening of Ganges Canal at Toghulpoor.

The Eastern Jumna Canal,† having a discharge of 1,068 cubic feet per second, had, when originally constructed, a grade of 372 feet in 130 miles, or at the average rate of 2.86 feet per mile. In some reaches the grade was considerably more than this.

Immediately after the water was admitted into the canal the effect of a rapid current was apparent. Retrogression of levels on an extensive and dangerous scale took place. From the Nowgong Dam to the Muskurra River, where the fall was eight feet per mile, deep

* Report on the Ganges Canal by Captain J. Crofton, R. E.

† Notes and Memoranda on the Eastern Jumna Canal, by Colonel Sir P. T. Cautley, K. C. B.

erosion took place. A specimen cross-section on this reach is given in Figure 128, where the shaded portion shows the part scoured out. The silt resulting from the erosion was carried down the canal until it got to a level reach, where it was deposited, causing the bed and banks to silt up, as shown in Figure 129. The shaded part shows the silting up; the top of the bank and outside slope are shown dressed up.

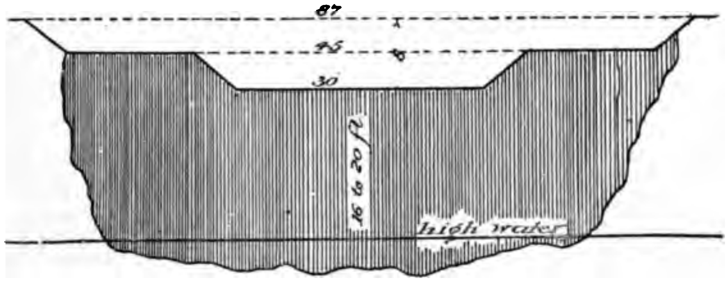


Fig. 128. Erosion on Eastern Jumna Canal.

To prevent the canal from being destroyed it had to be reconstructed at great expense.

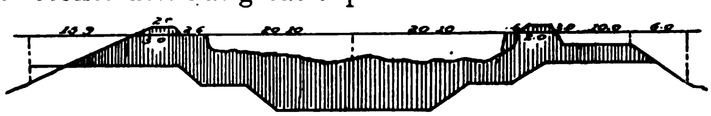


Fig. 129. Silting up on Eastern Jumna Canal.

Twenty-three falls were constructed, being at the rate of about one fall to six miles of canal, and the grade was reduced, varying from 17 to 22 inches per mile.

Article 39. Falls—Drops—Checks.

In designing an artificial channel for the passage of a large volume of water, the first thing that presents itself for decision is the rate of slope that is to be given to the bed, to insure that velocity of current which prevents the deposition of silt, keeps the channel clear of weeds and other impediments, and, at the same time, shall not erode the bottom and sides of the channel.

When the slope or grade of the canal is the same as the natural fall of the country through which the canal is excavated, and when the current is adjusted, as above explained, to prevent silting up and erosion, then the level of its bed will, of course, remain at a uniform depth below the surface of the ground.

Bed of Canal in Embankment.

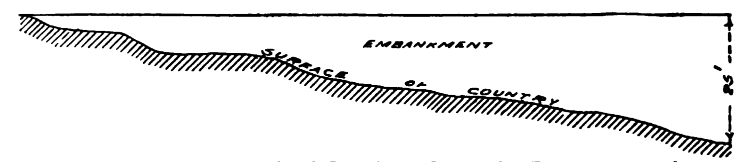


Fig. 130. Longitudinal Section, Canal in Embankment.

Usually the slope of the country is greater than that of the canal, and, with canals having a large discharge, that is, from 1,000 to 6,000 cubic feet per second, this is invariably the case, and the excess of slope of the country has to be disposed of, either by embankments or by works variously called falls, drops, and sometimes in America, checks.

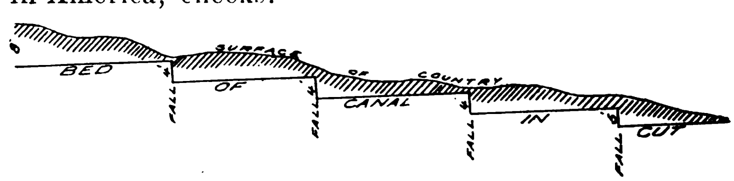


Fig. 131. Longitudinal Section, Falls in Canal.

In brief, then, the object of falls is to get rid of a greater declivity of bed than it is advisable to allow in mere earthen channels, and it is sought to be attained by giving at intervals sudden falls protected by masonry, between which the simple earthen bed may preserve its proper slope.

Figure 130, not drawn to scale, shows a reach of a canal in embankment, five miles in length. In this five miles the grade of the country has gained 25 feet on that of the canal, and, it is obvious, that an em-

bankment is out of the question on account of its great cost, the danger of breaches in its banks and several other good reasons. To compensate, therefore, for the difference of slope, *falls* are constructed on irrigation canals, as they are safer and cheaper than embankments.

Figure 131, not drawn to scale, shows how the falls are arranged so that the canal is, either in whole or in part, in soil. The canal is laid out in a series of *steps*, so as to keep it at a tolerably uniform level below the surface of the country, until the flat country is reached. By this time, the supply of the canal is diminished, and it, therefore, requires a greater slope to keep up the original velocity, and usually a point will be reached where the slope of the country is the same as that fixed for the canal.

When designing an irrigation canal, a minimum depth of excavation is determined, and then, when the depth of cutting becomes less than this, it is time to locate a fall.

The shape and construction of falls are questions requiring much thought and consideration. Their location should evidently, from the diagrams, Figures 130 and 131, be near the places where the canal bed, if continued without a break, would have to be carried in embankment above the surface of the country. Their *exact* location is generally made to coincide with the requirements of a highway bridge, regulator, or some other masonry work, such as are herein described, for the sake of economy, or for some other good reason.

In America, falls are usually constructed of timber, and they have not only the disadvantage of being built of perishable material, but they have also other defects, the chief of which is the great velocity of the water at and near them, which often causes their destruction.

Outside of America, in India, Italy and other irrigating countries, the falls are permanent works, constructed of brick or stone masonry. On the best works the banks of the canal, both above and below the fall, are protected from the erosive action of the water.

Six descriptions of falls are in use:—

1. The Ogee Fall.
2. The Vertical Fall, with water cushion.
3. The Vertical Fall, with gratings attached.
4. The Vertical Fall, with sliding gates.
5. The Vertical or Sloping Fall, with plank panels or flash boards.
6. Rapids.

Rapids are described in Article 40.

Ogee Falls.

There has been much difference of opinion with reference to the exact shape of the fall. *Ogee* falls, similar to that shown in Figure 132, were adopted by Sir Proby Cautley, on the Ganges Canal, with the view of delivering the water at the foot of the fall as quietly as possible.

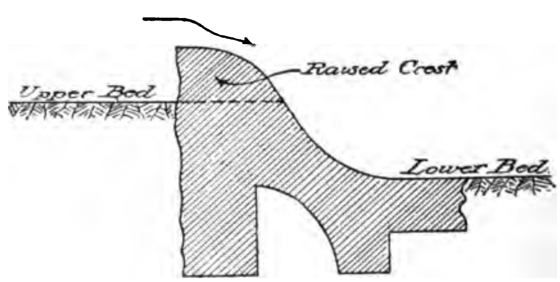
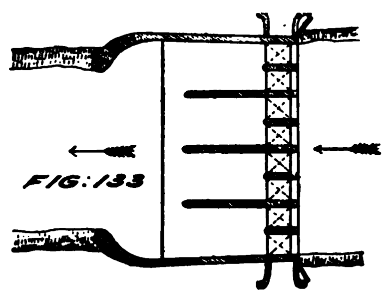


Fig. 132. Section of Ogee Falls.

The following is a description of one of the Ogee falls on the Ganges Canal: Figure 133 shows a plan, and Figure 134 a view of the Asufnuggur Fall on the Upper

Ganges Canal. This fall is shown attached to a bridge. The bridge consists of eight spans of 25 feet in width each, which crosses the canal on the upper levels. To the tail or apron of this bridge the ogees are attached, delivering the water into four chambers of 54½ feet in width, every alternate bridge-pier being prolonged on its down-stream face, so as to divide the space, which is occupied by the lower floorings, into four compartments. In advance of the three dividing walls, which are carried to a distance of 84 feet from the down-stream face of the bridge, there is an open space of masonry flooring, which is protected by an advanced area of box-work, or heavy material filled into boxes or crates, and covered with sleepers, so as to retain the material in position.



Plan of Asufnuggur Falls.

Additional defenses are given these floorings by lines of sheet piling. The flanks of the chambers below the descent are protected by revetments, equal in height to the dividing walls. Between and on the flanks of these two jetties, lines of piles and other protective arrangements are distributed, so as to secure the safe passage of the water over the floorings, and to admit of the currents escaping from the works with as little tendency to danger as possible. The Ogee Falls have proved failures, both on the Upper Ganges and Baree Doab Canals. Col. Crofton, in his Report on the Ganges Canal, states

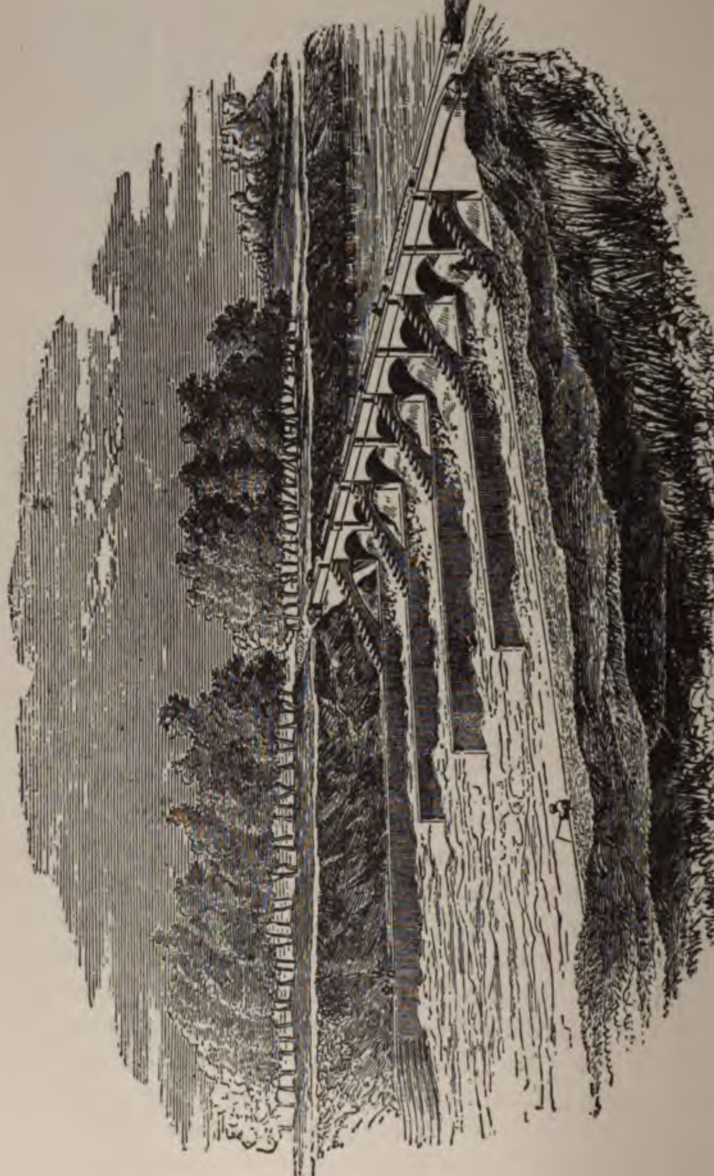


Fig. 134. View of Asufnuggur Falls, Original Ganges Canal.

that the greater number of the Ogee Falls on this canal suffered injury more or less severe, in their lower floorings from the action of the water, and in one or two cases the brick, on edge covering to the Ogees was stripped off, but timely repairs and protection saved the evil from spreading.

Vertical Falls with Water Cushions.

Vertical Falls with water cushions are illustrated in Figures 135 to 140 inclusive.

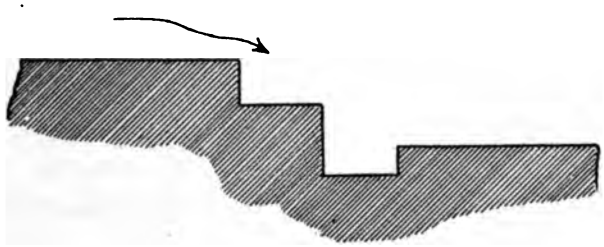


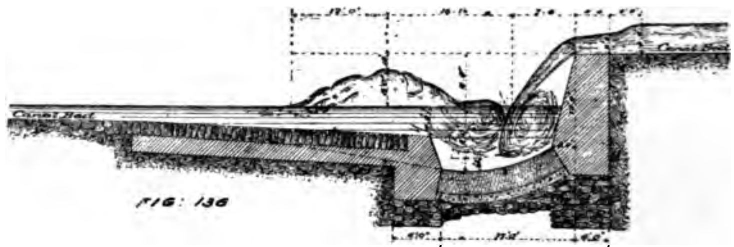
Fig. 135. Section of Vertical Fall.

These falls have been found much safer than the Ogee Falls. On the Baree Doab Canal, and generally on the new canals in India, Vertical Falls are used. These falls have a cistern on the lower side, and this cistern acts as a water cushion, and opposes a dead resistance to the falling water. The velocity of the falling water in a forward direction is also checked.

To lessen the destructive action of the falling water Mr. T. Login, M. I. C. E.* secured a framework of timber about five feet in height, above the crown of the Ogee Falls, instead of trusting to sleepers which were constantly giving way. By this arrangement the water was held up, so that erosive action on the bed and banks

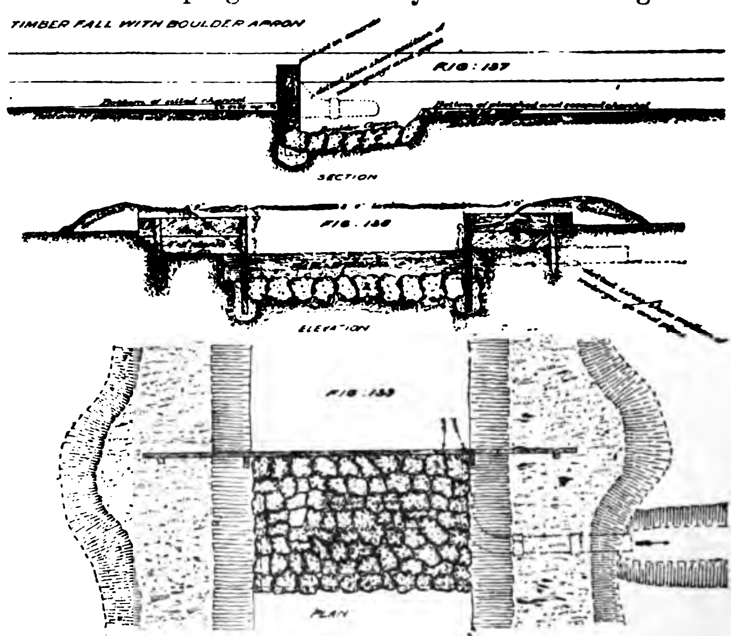
*On the Benefits of Irrigation in India, and on the proper Construction of Irrigation Canals by Thomas Login, M. Inst. C. E., in Proceedings of the Institution of Civil Engineers, Volume XXVII.

was prevented on the up-stream side of the canal, and it is a remarkable fact, that though there was a perpendicular fall of five feet or more on the crown of the Ogee,



Section of Vertical Fall on Baree Doab Canal.

no injury was done to the brickwork at the point where the water impinged. Probably this was owing to the



water, which passed through the open spaces of the timber framework, forming a cushion for the descending water.

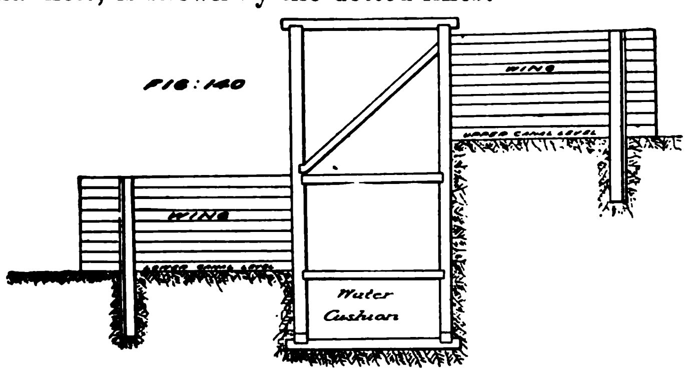
Mr. Login also constructed a rough sort of submerged weir 3½ feet high across the chambers of the falls, by which a cistern was formed to receive the descending mass, and in this manner diminished the destructive effect of the falling water.

Figure 136 shows a vertical fall with water cushion on the Baree Doab Canal, India. It will be seen that the bed is protected for some distance, on the lower side, with masonry and paved flooring.

Figure 137 shows a longitudinal section, Figure 138 a cross-section, and Figure 139 a plan of a vertical fall with water cushion, constructed of timber and bowlders, on a small irrigation canal on the Canterbury Plains, New Zealand.*

The maximum discharge of this canal is about 50 cubic feet per second. A water gauge, for delivering the required quantity of water to one of the distributing channels, is shown by the dotted lines.

FIG: 140



Section of Timber Fall with Water Cushion.

Figure 140 shows a vertical fall with water-cushion on the Turlock Canal, California. This diagram is taken from a paper by Mr. H. M. Wilson, C. E., in Transactions of Am. Soc. C. E., Vol. XXV.

* Water Supply and Irrigation of the Canterbury Plains, New Zealand, by George Frederick Ritso, Assoc. M. Inst. C. E., in Proceedings of the Institution of Civil Engineers. Volume LXXIV, 1883.

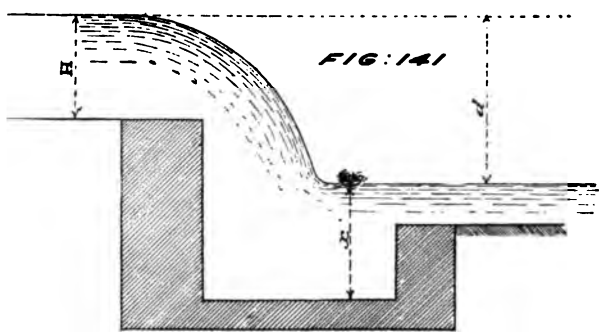
The following formula has been used in India to find the depth of the cistern below the lower bed of the canal. It is:—

$$x = (h)^{\frac{1}{2}} \times (d)^{\frac{1}{3}}$$

in which $x =$ the required depth of cistern below the lower bed of the canal.

$h =$ the height or fall, that is, the difference of level between the surface of water above the fall and the surface of water below it.

$d =$ full supply depth of water in the channel.



Section of Vertical Fall with Water Cushion.

It has been stated that all the cisterns constructed with depths thus obtained, have answered admirably, having required but slight repairs since they were built.

The very dangerous scouring and cutting action of a large body of water falling over a height of even a few feet can be readily understood. The greater the height of the fall and the depth of water, the more violent, of course, will be the action. Those on the original Ganges Canal are not higher than eight feet, but the destructive action of over 6,000 cubic feet of water per second, and having a depth on the crest of the fall of six feet or more, is very great, and nothing but the best masonry is capable of resisting it.

If stone of good quality can be obtained it should always be employed, laid on an unyielding foundation, with fine mortar joints. The banks must be protected with masonry for a considerable distance down stream, and the bed of the canal protected by a solid masonry flooring, the down stream end of which is protected by a row of sheet piling.

The depth of water over the crest of a fall is less than that in the canal above the fall, and it follows that the effect of a fall occurring at the end of a canal reach is to increase the grade, and, therefore, the velocity, and to diminish the depth of water for a considerable distance above the fall. The increase of velocity and diminution of depth are gradual from the point where the action commences down to the fall itself, where, of course, they attain a maximum, so that the depth of water passing over the fall is very much less, as the velocity is very much greater, than the normal depth and velocity above. This increase of velocity, before the water reaches the fall, produces a dangerous scour on the bed and banks of the canal, and in order to guard against this, it has been found necessary to head up the water at the falls on the Ganges Canal by means of sleepers dropped in the grooves of the piers, which has virtually increased the height of the fall, and has been one cause of the flooring, on the lower side of the fall, suffering in places from the violent action of the water. It has also been proposed to narrow the falls to produce the same effect.

The method most commonly adopted in India is to raise the crest of the falls by a masonry weir, as shown in Figure 132. At first the crest of the fall was on the level of the bed of the canal on the upper reach. The height to which it is necessary to raise the crest of the weir may be found from the following investigation, as

given by Colonel Dyas, modified, however, by the writer to suit the symbols used, and also Kutter's formula, as simplified in this work.

v = mean velocity in feet in open channel.

a = sectional area of open channel in square feet.

r = hydraulic mean depth of same in feet.

s = sine of slope.

h = height in feet of surface of water in channel, above crest of fall.

l = length of crest of fall in feet.

m = co-efficient of discharge over weir varying from 2.5 to 3.5.

c = co-efficient of discharge of open channel.

Allowing for velocity of approach, we have discharge over fall *complete*, that is, a free fall:—

$$Q = m\,l\left(h + \frac{v^2}{2g}\right)^{\frac{3}{2}}$$

but $v = c \times (r\,s)^{\frac{1}{2}}$ $\therefore$ $v^2 = c^2 \times rs$

substituting value of v^2 we have:—

$$\text{Discharge over fall } Q = m\,l\left(h + \frac{c^2\,r\,s}{2g}\right)^{\frac{3}{2}}$$

The discharge in channel above weir:—

$$Q = a\,c\,(r\,s)^{\frac{1}{2}}$$

$$\therefore \quad m\,l\left(h + \frac{c^2\,r\,s}{2g}\right)^{\frac{3}{2}} = a\,c\,(r\,s)^{\frac{1}{2}}$$

$$\therefore \quad m\,l\left(\frac{2\,g\,h + c^2\,r\,s}{2g}\right)^{\frac{1}{2}} \times \left(\frac{2g\,h + c^2\,r\,s}{2g}\right) = a\,c\,(rs)^{\frac{1}{2}}$$

$$\therefore \quad l = \frac{1}{m} \times a\,c(rs)^{\frac{1}{2}} \times \left(\frac{2g}{2g\,h + c^2\,r\,s}\right)^{\frac{1}{2}} \times \left(\frac{2g}{2g\,h + c^2\,r\,s}\right)$$

$$\therefore \quad l = \frac{1}{m} \times 2agc \times \left(\frac{2grs}{2gh + c^2rs}\right)^{\frac{1}{2}} \times \frac{1}{2gh + c^2rs}$$

$$\therefore \quad l = \frac{1}{m} \times 2agc \times \frac{(2grs)^{\frac{1}{2}}}{(2gh + c^2rs)^{\frac{3}{2}}}$$

Now, to find h, we have from equation above by squaring,

$$m^2 l^2 \left(h + \frac{c^2 rs}{2g}\right)^3 = a^2 c^2 rs$$

extract cube root

$$(m^2 l^2)^{\frac{1}{3}} \times \left(\frac{2gh + c^2rs}{2g}\right) = (a^2 c^2 rs)^{\frac{1}{3}}$$

$$\therefore \quad \frac{2gh + c^2rs}{2g} = \left(\frac{a^2c^2rs}{m^2l^2}\right)^{\frac{1}{3}}$$

$$h + \frac{c^2rs}{2g} = \left(\frac{a^2c^2rs}{m^2l^2}\right)^{\frac{1}{3}}$$

$$\therefore \quad h = \left(\frac{a^2c^2rs}{m^2l^2}\right)^{\frac{1}{3}} - \frac{c^2rs}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad (A)$$

Having thus got the value of h, deduct it from the depth of water in the channel, and we have the height to which the weir should be raised above the bed of the canal, Figure 132, in order that the water in its approach to the weir may not have any increase of velocity.

Example:—Let the bed width of the channel above the weir be 60 feet, depth 9 feet, side slopes 1 to 1, grade 6 inches per mile and $n = .025$. Also, let the length of the crest of weir be 55 feet. Now, let us compute to what depth the crest of the weir must be raised, in order that the water approaching the weir may not have a greater velocity than the mean velocity in the open channel.

In the open channel we have: (see *Flow of Water*)

$$r = \frac{a}{p} = \frac{621}{84.726} = 7.3295 \text{ and } \sqrt{r} = 2.7 \; m = 3,$$

and $c = 86.4$

$s = 6$ inches per mile $= .000094697$;

substituting values of a, c, r, s, m, l, and g, in above formula (A) we have

$$h = \left(\frac{621^2 - 86.4^2 + 7.3295 \times .000094697}{3^2 \times 55^2}\right)$$

$$\left(\frac{86.4^2 \times 7.3295 \times .000094697}{2 \times 32.2}\right) = 4.1808 \text{ feet.}$$

Now, as a check, let us compute the discharge over weir:—

$Q = 3 \times 55 \times 4.1808 \sqrt{4.1808}$

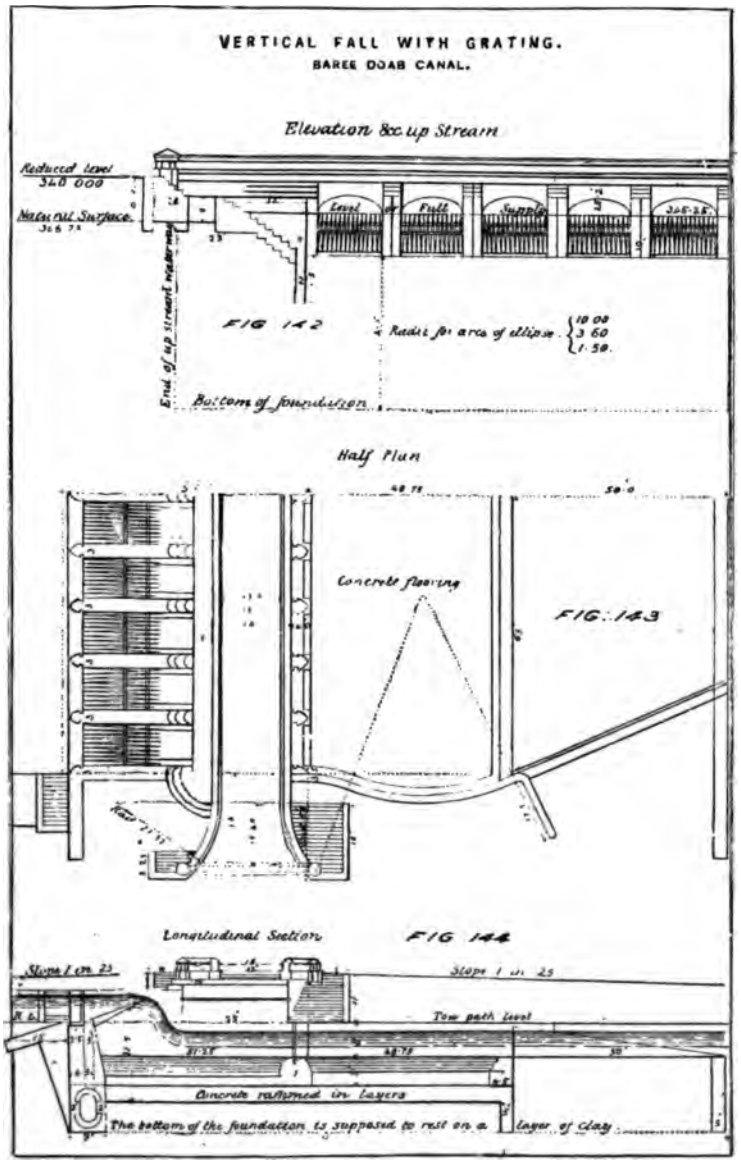
$= 1409$ nearly, which is also the discharge in the open channel, computed by Kutter's formula, with the dimensions above given. Now $9-4.1808=4.8192$ feet is the height above crest of fall to which the weir must be built. This raising of crest, however, is suited to only one depth of water in the open channel. A much better plan by which the crest of the weir can be adjusted to any depth of water in the channel, is shown in Figures 152, 153 and 154.

Vertical Falls, with Gratings.

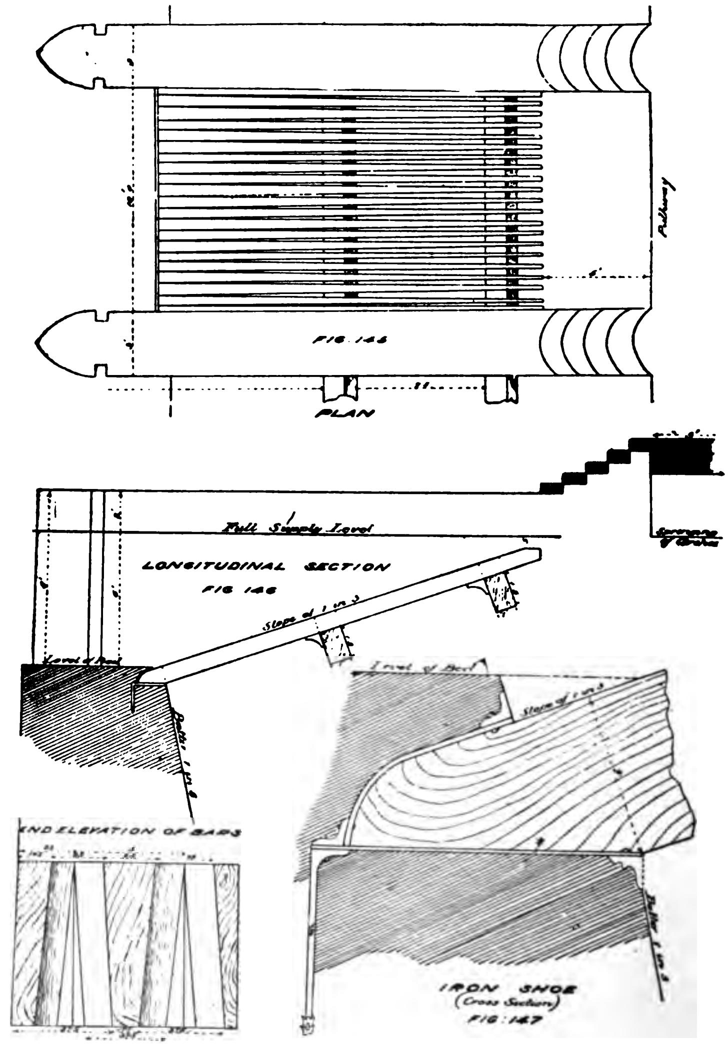
The result of experience seems to show that Vertical Falls with Gratings, as used on the Baree Doab Canal, and illustrated in Figures 142 to 151 inclusive, are the best that have yet been adopted.

By referring to the drawings, it will be seen that the water is made to fall vertically through a grating laid at a slope of about one in three, and that its action on the surface below is thus spread over as large an area as may be wished. Owing to the several filaments of water

being separated by the bars, much air is carried down with the water, and the action below is reduced to a



minimum. The bars are laid longitudinally with the stream, and at their lower ends, which rest on the crest



of the fall, they are close together, and at the upper end they are farther apart. The teeth of a comb give a good idea of the arrangement.

The grating consists of a number of wooden bars resting on an iron shoe built into the crest of the fall, and one or more cross-beams, according to the length of the bars. They are laid at a slope of one in three, and are of such length, that the full supply level of the water in the canal, tops their upper ends by half a foot.*

The grating divides the water into a number of filaments or threads, and spreads the falling volume over a greater area, thus lessening very much its destructive action on the floor of the cistern.

The scantling of the bars as well as of the beams should, of course, be proportioned to the weight they have to bear, plus the extra accidental strains to which they are liable, from floating timber for instance, which may possibly pass between the piers and so come in contact with the grating. In consideration of strains and shocks of this nature, the supporting beams are set with their line of depth at right angles to the bars instead of vertically.

The dimensions of the bars used on the falls of the Baree Doab Canal, where the depth of water is 6.6 feet, are as follows:—

Deodar Wood.

Lower end of bars, $0'.50$ broad $\times$ $0'.75$ deep,

Upper end of bars 0.25 broad $\times$ 0.75 deep,

and they are supported on two deodar beams, each measuring one foot in breadth $\times$ 1.5 feet in depth; the first beam being placed at a distance of 7.5 feet (horizontal measurement) from the crest of the fall, and the second 7.5 feet beyond the first beam.

The bars of the grating on these falls were originally placed touching each other, side by side, at their lower

*Captain J. H. Dyas in Professional Papers on Indian Engineering, Volume 3. First Series.

ends, as there was not then a full supply of water in the canal. There were thus 20 bars in each 10-feet bay. Since then the number of bars has been necessarily reduced to 19 and to 18, the latter being the present number. The reduction of the number of bars and the equal spacing of the remaining bars is done with ease, as they can be pushed sideways in the iron shoe and along the beams, to which latter they are held with spike nails. Once the correct spacing is arrived at, cleats and blocks are preferable to spike nails.

The bars are undercut from the point where they leave the shoe, *i. e.*, from the crest of the fall, so as to make each space, as it were, "an orifice in a thin plate," and it facilitates the escape of small matters which may be brought down with the current. Large rubbish, which accumulates on the grating, is daily raked off and piled on one side of the fall. This is done by the establishment kept up for the neighboring lock. There is considerable advantage in thus clearing the canal of rubbish, which would otherwise stick in rajbuha (distributary) heads, on piers of bridges, etc., or eventually ground on the bed of the canal, and become nuclei of large lumps and silt banks. But supposing that there were no one at hand to rake the *debris* off, and that the grating became choked, the water would merely rise until it could pour over the top of the grating, and the rubbish would be swept over with it.

Where gratings are used they act instead of a weir in checking the velocity of the water above the falls, and the principle to be adopted in spacing the bars, is to arrange them so that the velocity of no one thread of the stream shall be either accelerated or retarded by the proximity of the fall. This effected, it is evident that the surface of the water must remain at its normal slope, parallel to the bed of the canal, until it arrives at the grating.

To take an example, let us assume that:—

mean vel. $v = 0.81\ v_{max}$

$v_b = 0.62\ v_{max}$ (in every vertical line of the current flowing naturally.)

where v = mean velocity in feet per second,

v_{max} = surface velocity in feet per second,

v_b = bottom velocity in feet per second.

Then if we make $v = 2.5$ feet per second, we shall have the following velocities at the given depths below the surface in a stream 6 feet deep —

Depths below surface in feet.	Velocities in feet per second	REMARKS.
Surface..........0	3.0864	
" " 1	2.8909	
" " 2	2.6955	
Center...........3	2.5	Common difference 0.1955 nearly.
" " 4	2.3046	
" " 5	2.1091	
Bottom..........6	1.9136	

What is required, then, is to shape the sides of a given number of bars, placed in a given width of bay, so that the above velocities may be maintained till the water touches the grating, when, in consequence of the clear fall, the velocity becomes considerably accelerated. This accelerated velocity multiplied by the reduced area, of space between the bars, should give the same discharge, with the canal running full, as the product of the original normal velocity and the original undiminished space, the width of which is, of course, the distance between the centers of two contiguous bars.

Thus, taking the lowest film, along the bed of the canal, whose normal velocity is 1.9136 feet per second, and supposing 20 to be the number of bars in each bay, then the undiminished space for each portion of the stream will be half a foot, which multiplied by the above velocity gives a product of 0.9568. Again, taking, the same lowest film as it passes through the grating, with a clear fall, and under a head of pressure of six feet, we find its velocity to be 19.654 feet per second. Now, if we called the required width of space between the bars at this point x_a, and assume the co-efficient of contraction to be 0.6, we shall have:

$$x_a = \frac{0.9568}{19.654 \times 0.6} = 0.08 \text{ foot.}$$

Similarly, taking the film on the level of the tops of the bars, or 0.5 foot below the surface of the water, the normal velocity of which is 2.9887, the undiminished space being, as before, 0.5 foot, we get a product of 1.4944; and as the velocity of the film falling through the bars is 5.673 feet per second, we get:—

$$x_s = \frac{1.4944}{5.673 \times 0.6} = 0.44 \text{ foot.}$$

And lastly, taking the center film, the normal velocity of which is 2.5 feet per second, we have a product of 1.25, and as the velocity of the same film passing through the grating is 13.89 feet per second, we get:—

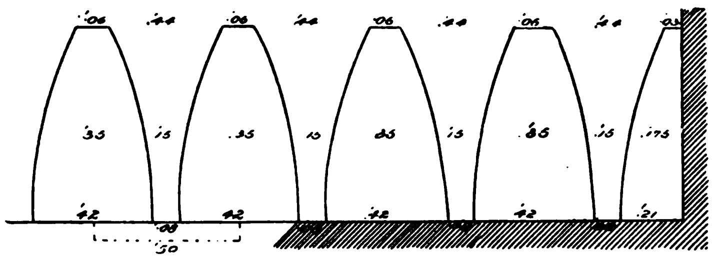
$$x_t = \frac{1.25}{13.89 \times 0.6} = 0.15 \text{ foot.}$$

Hence, it is seen that the sides of the bars should be cut to a curve, as shown in Figure 149, convex towards the open space; but in practice this nicety is scarcely requisite, and they may be made as shown in Figure 150.

The above remarks have been limited to a consideration of the effect caused by the grating on the channel

above the fall. Its effect on the channel *below* the fall is equally important. The velocity, eddies and consequent erosion below the fall are much diminished by the gratings.

Fig. 149.



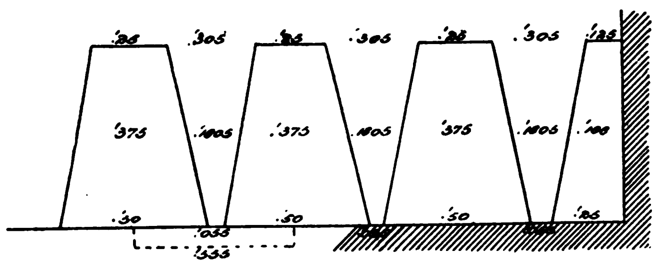


Fig. 150. Plans of Bars of Grating.

Colonel Dyas, an engineer of great experience, when in charge of the Baree Doab Canal wrote on this subject*:—

"In my opinion the Grating Fall is the best fall yet known, and the next best is the Vertical Fall without grating. We have but one Ogee Fall on the Baree Doab Canal, and that one has given us more trouble in repairing it than all the rest together. Indeed we have not had to touch the others although we have had a flood down the canal that submerged them. You can have no idea without seeing them how completely under control the water is by their means. *Divide and conquer* is their motto, and I think it is the true principle."

*Professional Papers on Indian Engineering, Volume I. First Series.

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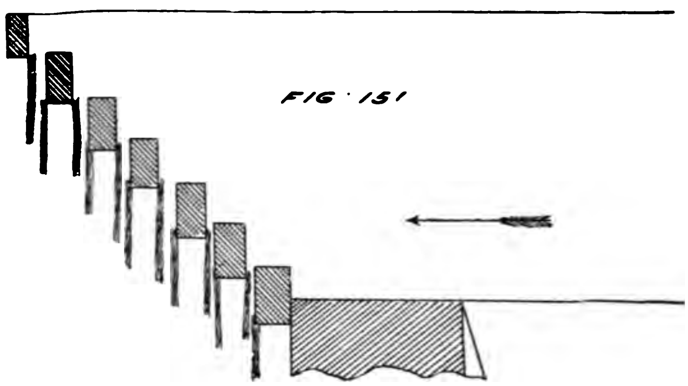
Figure 151 is a cross-section of a grating having horizontal bars at right angles to the axis of the canal.



Section of Grating having Horizontal Bars.

Fall with Sliding Gate.

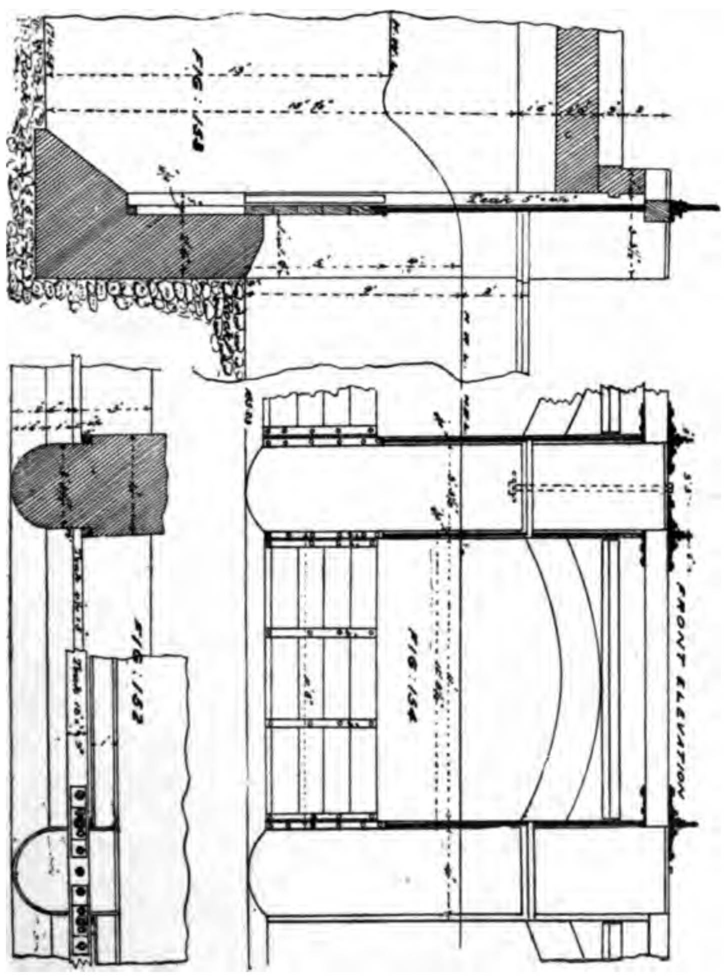
A Fall with a sliding gate, on the Sukkur Canal, in India, is shown in Figures 152, 153 and 154.*

Above the fall the canal has a bottom width of 60 feet, depth 9 feet, side slopes 1 to 1, and a fall of 6 inches per mile. The mean velocity is 2.27 feet per second, and the discharge 1,410 cubic feet per second. Kutter's formula, with a value of $n = .025$, applied to this channel, will give the mean velocity mentioned of 2.27 feet per second. See the *Flow of Water*.

The fall is divided into five bays of 11 feet each in width, by piers 4 feet thick. The plan of one of these bays is shown in Figure 152. The difference of level between the beds of the canal above and below the fall is 7.55 feet, and of the high water lines 3.55 feet. The crest of the masonry portion of the weir is nine inches above the bed, Figure 153.

* Col. J. Le Mesurier, R. E., in Professional Papers on Indian Engineering, Vol. 5, Second Series.

The thickness of the weir is two feet six inches. It is in fact nothing more than a brickwork facing to the rock, forming an even surface, on which the timbers are fixed, against which the gates can slide.



Fall on the Sukkur Canal, India.

There is no cistern or basin to form a water-cushion under the falling water, as the bed at this place is com-

posed of sound rock. The bed and banks, for a distance of 400 feet below the falls, are protected with rough stone pitching, laid dry, about one foot six inches or two feet thick.

The plan of using *sliding gates* to form the weir, instead of building up a mass of masonry above the bed, is believed to have been introduced for the first time on the Sukkur Canal, to regulate the depth of fall to actual discharge on a canal with a maximum capacity of over 1,400 cubic feet per second.

The gate, Figures 153 and 154, is constructed of four-inch teak plank, with a strip of 3½-inch angle-iron along the top and bottom of the down-stream face. The gate is strengthened, at front and back, by four strips of three-eighths inch plate iron four inches wide, and by two cross-pieces of 3½-inch angle-iron at the back. The gate, when lowered to the full extent, rests on a piece of teak 11′ 8½″ × 5″ × 4¼″, fastened to the brickwork by bolts, and its top is then level with the crest of the masonry, or nine inches above the bed of the canal. It slides up and down against two vertical straining pieces of teak scantling 5″ × 4¼″, fastened by lewis bolts to the piers, which are recessed for the purpose; the thickness of the pier being four feet, and of the upper cutwater three feet three and one-half inches.

When the full supply is going over the gate its top is five feet above the level of the bed, or its bottom nine inches below the crest of the masonry. The man in charge of the falls has orders to keep the gauges at the head regulator and at the falls reading the same, and when this is the case, the surface slope of the water is six inches per mile. If less than nine feet is admitted at the head, the gates at the falls are lowered until the two gauges read the same. If at any time it is necessary to admit a greater depth than nine feet, the gates are raised.

The apparatus for raising or lowering the gates is very simple. Across the cutwaters, a teak beam nine inches wide by twelve inches deep, is laid and bolted down to the piers by a two-inch bolt. The screws which are attached to the gates are of two-inch rod, cut to one-quarter inch pitch; they pass through holes cut in the teak beams, and are wound up and down by a brass nut, turned by an iron handle. In the cold weather, when the canal is dry, the wood and iron work of the gates are well dressed with common fish oil, procured from the fishermen on the river.

The gates are eleven feet eight inches long, and as the opening in which they slide is eleven feet eight and one-half inches, they have a play one-quarter inch at each end. There is also a small play between the front of the gate and the back of the masonry of the weir wall; one-quarter inch is shown in Figure 153, but it is in reality less than this. The four-inch strips of plate iron are countersunk into the front of the gate, but not into the back, and all the rivets and bolts as well, so that the face of the gate is perfectly level and flush; and there is no reason why more than one-sixteenth of an inch play should be given. It was considered advisable, however, as the gates had to be made in Karachi, and sent up to Sukkur ready to be put up, to allow for one-quarter inch play when building the masonry.

One advantage of this kind of fall, and a very great one, is that it suits a variable depth in the canal, as the gate can be raised or lowered, according to the depth of water admitted. Another advantage appears to be that, the action of the water upon the bed and banks below the fall is reduced to a minimum. The canal is merely protected by a comparatively thin layer of rough stones, procured from the excavation, and laid dry, and up to the present time no repairs of any sort have been re-

quired. The bed and banks of the canal above the falls are almost as clean as the day they were cut, as, whatever the depth of water is, the surface slope is kept fixed at six inches a mile, and the mean velocity never exceeds two and one-quarter feet per second, which is the velocity with maximum supply.

Fall with Plank Panels or Flash Boards.

Figure 155 shows a timber fall on the Calloway Canal, California. Flash boards can be placed on the framing of this fall to regulate the height of the water in the upper reach of the canal. This is a very light structure, and it is built on a somewhat similar plan to that of the Kern River Weir, Figure 20.

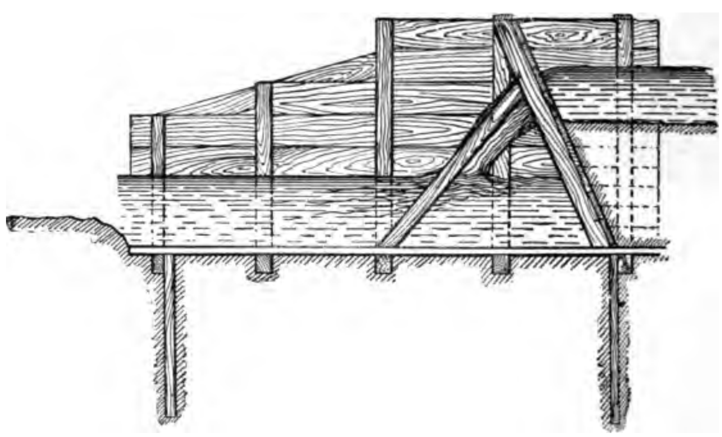


Fig. 155. Timber Fall with Plank Panels or Flash Boards.

A good floor for the lower part of drops is sometimes made in the following manner: A wooden box is constructed as large as the intended floor, and from one to three feet in depth. If the material is sand or loam, the joints of the boards are covered inside with thick tarred brown paper, after which the box is filled with sand or loam and the cover nailed on. The filling gives the box weight and stability.

At the lower edge of the box a row of sheet piling is sometimes fixed. The sheet piling should not be driven. The best way to fix it is to excavate a trench and, if not too large, to frame the sheet piling together and put it into the trench framed in one piece, then fill in the material on each side and tamp it in layers. A piece of timber can then be fixed and spiked to the top of the sheet piling and the box.

On the Uncompahgre Canal in Colorado, carrying 725 cubic feet per second, the water drops 230 feet over a precipitous rocky cliff into the bed of a dry wash.

Article 40. Rapids.

Instead of falls, and to accomplish the necessary change of level, *Rapids* have been employed with success on the Baree Doab Canal,* that is, the fall is laid out on a long slope of about 15 to 1, instead of by a single drop. The slope is paved with bowlders, laid with or without cement, and confined by walls of masonry in cement, at intervals of 40 feet, both longitudinally and across stream. The longer and flatter the slope, the more gentle is, of course, the action of the water; but the greater, also, is the quantity of masonry employed. In general the choice between the two is a mere question of expense and material available. On the Baree Doab Canal, rapids were adopted wherever bowlders were procurable at moderate cost. Figures 156, 157 and 158 show a rapid on the Baree Doab Canal.

Bowlders or quarry stone are the proper material for the flooring of a rapid, and soft stone or ordinary brick work should not be used in contact with currents of such high velocities. Even the very best brick work cannot

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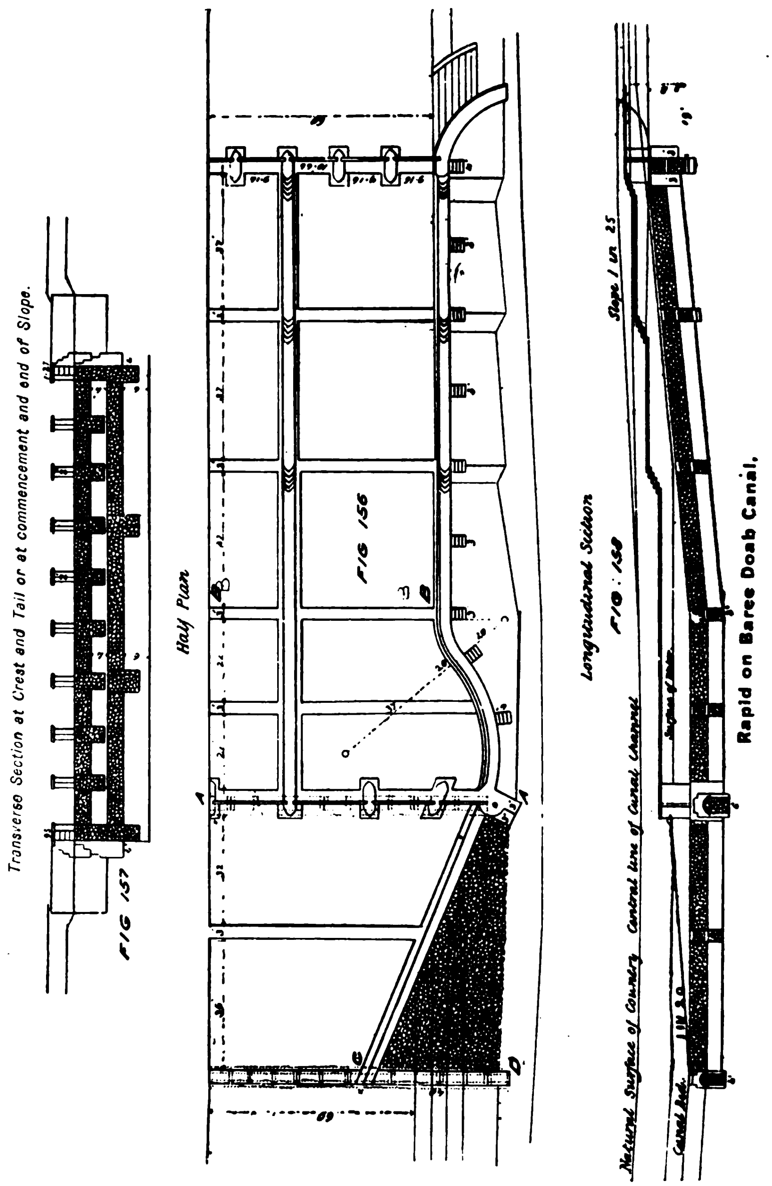
*Professional Papers on Indian Engineering, Vol. I, First Series, and Roorkee Treatise on Civil Engineering.



Rapid on Baree Doab Canal.

stand the wear and tear, for any length of time, of water at a high velocity and carrying sand and silt. Hard stone should be used with all surfaces in contact with velocities exceeding, say, ten feet per second.

The bowlders should generally be grouted in with good hydraulic mortar and small pebbles or shingle. Portland cement mortar, if available at moderate cost, would be the best cementing material. Dry bowlder work is not to be depended on for velocities higher than 15 feet per second, even when they weigh as much as 80 pounds each, and are laid at a slope of 1 in 15. There should be no attempt made to bring the surface of the bowlder work up smooth, by filling in the spaces *a*, *a*, *a*, Figure 159.

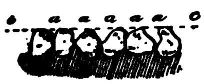


Fig. 159.

All that is necessary is to lay the bowlders, and to pack them, so that their tops are pretty well in line as *b*, *c*; any further filling in would stand a good chance of being washed out very soon, and if it remained, its effect would be to increase the velocity of the current on the rapid by diminishing the resistance presented to the water by the rough bowlder work.

The Baree Doab Canal Rapids have tail walls of peculiar construction, Figure 156, for the purpose of destroying back eddies, and of protecting the canal banks below the rapid from the direct action of the current. These tail walls are intended to be so arranged that the heaviest action of water at the foot of the rapid shall take place in the widest part *AA*, the normal width of the rapid being represented by *BB*, and they incline towards each other from this point so as to direct the set of the stream well to the center of the canal, thus protecting the banks from the direct action of the current for a considerable distance. At the same time, as may be seen from the longitudinal section, Figure 158, the tail walls are not

kept at their full height throughout, but beginning a little below where the curve ends, at the level of full supply only, they gradually become lower and lower, slope 1 in 20, till they vanish altogether, where they are on the same level as the bed of the canal. The triangular spaces, *A C D*, behind the walls in plan, are filled in with dry bowlders, to the level of the top of the sloping tail wall. When the full supply is running, these tail walls are submerged and invisible, the rapid appearing to end just below *A A*. These tail walls do not check the "lap-lap" or ceaseless wave-like undulation of the water below the rapid. That is not their office, and indeed it would be difficult to check that movement, but they effectually do away with back eddies by keeping the current always in onward motion, exposing no abruptly terminating projection behind which an eddy can form, and at the same time they protect the banks by making that motion moderate in the neighborhood of the banks.

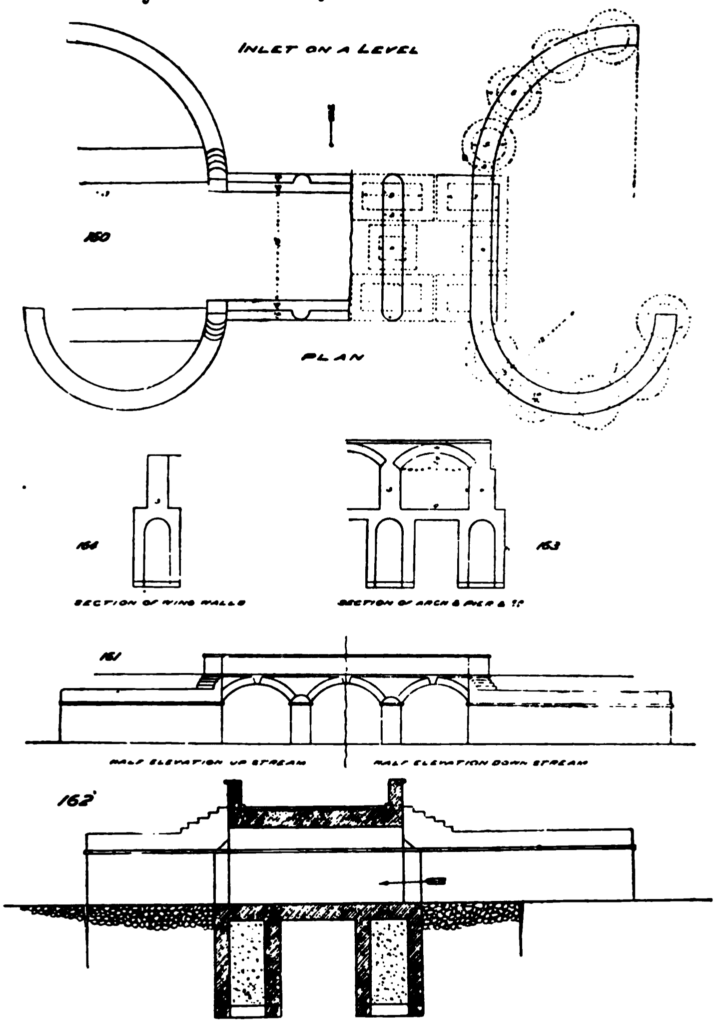
In case no such tail walls are given, experience has shown that the banks of the canal when constructed of ordinary loam, should be faced with bowlders or some other protection for a length of 300 feet below the rapid and on each side of the canal.

The maximum velocity of current which a bowlder rapid will stand without injury cannot be exactly determined, but experience has proved that a rapid, such as is shown in Figures 156, 157 and 158, with a flooring composed of bowlders, weighing not less than eighty pounds each, well packed *on end*, somewhat similar to Figure 159, and at a slope of 1 in 15, will *not* stand a mean velocity of 17.4 feet per second.

A good example of a wooden flume rapid has already been illustrated in Figures 87 and 88, page 153.

Article 41. Inlets.

When a canal crosses a small drainage channel that is filled only occasionally, in very heavy rains, and



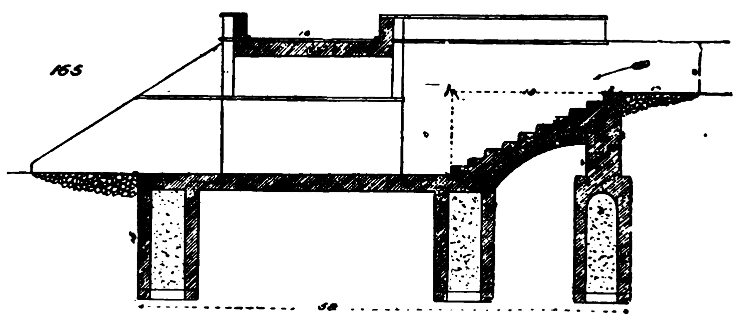
whose duration of flood lasts but a short time, an Inlet is provided in the canal embankment to allow the flood

water to pass into the canal. On the Indian and Italian canals, this inlet is usually also a bridge to keep communication open along the canal embankment.

Figures 160 to 164 show details of an inlet on a level,* that is, the level of the bed of the drainage channel is at, or nearly on, the same level as the bed of the canal. There are usually no gates to an inlet. Sometimes perennial streams, when they carry no debris or silt, are admitted into the canal by an inlet.

An inlet differs from a level crossing, shown at page 167, in so far as that it has not an outlet on the opposite side of the canal.

Figure 165 shows an inlet with 10 feet fall, from the



Section of Inlet.

bed of the torrent to the bed of the canal. To pass the torrent over the canal by a Superpassage, *Article* 36, or by an Inverted Syphon, *Article* 37, would be a very expensive work, therefore, an inlet was adopted as being by far the least expensive.

For small streams, cement pipe or vitrified stoneware pipe are very suitable as inlets.

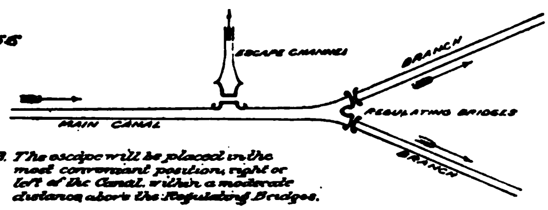
* Sone Canal Project, by Col. C. H. Dickens.

Article 42. Heads of Branch Canals.

On the first-class Indian Canals it is usual to place a Regulator, both on the main line and at the head of a

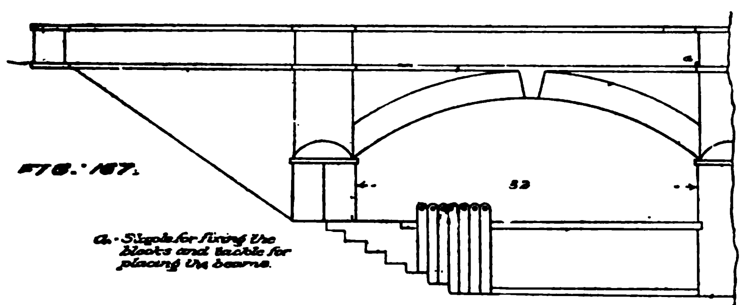
Diagram showing the relative position of the Regulating Bridges and Escapes at Branch Heads.

FIG: 166



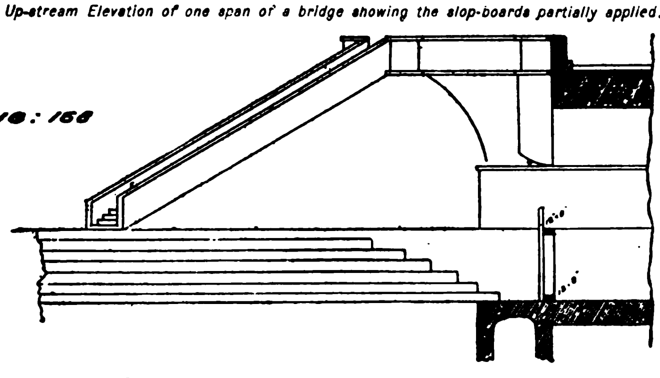
REGULATING BRIDGE ON BRANCH HEAD

FIG.: 167.



Up-stream Elevation of one span of a bridge showing the slop-boards partially applied.

FIG.: 168



Section of Bridge, half roadway, with slop-boards fixed.

branch canal, as shown in Figures 166 and 169. These regulators are usually combined with highway bridges constructed of masonry.

On the Sone Canals, India, the original plan provided for branch regulators, on the French Needle Dam plan, Figures 167 and 168, and an escape above each bifurcation, of sufficient capacity to lay both the lower channels dry, as shown in Figure 166. Where the object is to diminish the supply of water in both, it will be unnecessary to do more than open the requisite number of bays of the escape bridge. But when it is desired to keep up the whole supply in one channel, and reduce it, or altogether cut it off, in the other, it will be necessary to drop the sill beam in by the grooves, Figures 167 and 168, using the blocks and tackle, in the deep channels, for the ends near the pier, and afterwards to fix the beam in its seat by the same means. After this, using the upper beam as a bridge, the needles will be applied by hand, to such an extent as may be desired.

The plan will not be so expeditious as that of the drop-gates and windlasses shown in Figure 62. It will, however, provide in a simple way for all that is wanted for small regulators. By the use of a few long drop boards, let down from the parapet of the bridge, the openings could be partially closed without stopping navigation.

In Figure 167 a tow-path for convenience of navigation is shown under the bridge, and seven wooden needles in position to partly close the opening, and in Figure 168 the needles are shown in section, and masonry steps are shown leading from the bridge roadway to the canal.

The following details of the working of a needle dam on the Sidhnai Canal, India, are given here:—*

* The Sidhnai Canal System, by Loudon Francis MacLean, in Proceedings of the Institution of Civil Engineers, Volume CIII, 1891.

"The needles are made of deodar wood, and are seven feet six inches long, by five inches by three and one-half inches, with a stout handle 18 inches long, ending in a knob; they weigh 36 pounds dry and 40 pounds wet, and can be manipulated by one man. After placing the needles in position at first, they are forced up close together by a man standing on the pitching below the dam, who inserts a crowbar with a wedge-shaped end into the opening, causing the needles to slide along the face of the crest wall, any leakage between them being stopped in the following way: A basket fixed to a bamboo about 10 feet long, and filled with shavings or chopped straw, or some similar substance, is slipped down in front of the leak, so that the light material may be sucked by the current into the opening, which it effectually closes. It was not found that the shock of closing on the crest wall, when first placing the needles in position, ever caused them to break when the wood was sound.

"When there is a great difference of level between the water above and below the dam, a rush of water through the interstices makes it very difficult for a man to stand on the pitching below and use a crowbar. The difficulty is overcome in the following way: A piece of tarpaulin or oiled canvas, eight feet long and six broad, is fastened at one end to a wooden bar six feet four inches long, with handles at each extremity, and at the other end to a bar of round iron six feet four inches long and one inch in diameter. It is then rolled upon the iron bar, and placed horizontally against the needles, above where the excessive leakage occurs, and the wooden bar, which remains on the outside of the roll, is either tied or held in position by the handles; the roll is then let go, and the weight of the iron bar causes it to unroll itself down the face of the needles, at once

closing all the leaks. In order that the screen may be more easily recovered, a cord is attached to a loose collar at each end of the iron bar, and when the needles have been closed up, the screen is pulled up from the bottom by these cords.

' For the purpose of regulating the height of water above the dam, it is sufficient in most cases to push some of the needles forward at the top, the water escaping through the open spaces left in this way; but should it be necessary to provide for a greater flow, a sufficient number of them are removed altogether. This can generally be done by hand, but if they have "jammed" from any cause, or if the pressure of the water against them is too great, they are lifted by means of a bent lever.

"An eye bolt is attached to each needle just below the handle; this serves as a fulcrum for the extracting lever, and also to fasten tackle to when the pressure is too great for the needles to be drawn forward by hand. It was found dangerous to work them from the beams, which are only 18 inches wide, and after one life had been lost, and the Author himself had a narrow escape, a foot bridge was added to the dam. * * * *

" Arrangements have been made to send warnings by telegraph of any rise of one foot in the Ravi at Madhopur and Lahore during twenty-four hours. As floods take a minimum of five days from the former, and two days from the latter place to reach Sidhnai, these warnings have been of the greatest service."

Figure 169 is a plan of the regulator at the head of the Kotluh branch of the Sutlej Canal, designed by Major J. Crofton.* With reference to this work he states:—

* Report on the Sutlej Canal.

"The Kotluh branch will take off at an angle of 45° from the main channel, the direction of the central line remaining unaltered. A water-way of 64 feet is given to the central line, and 50 feet to the Kotluh branch,

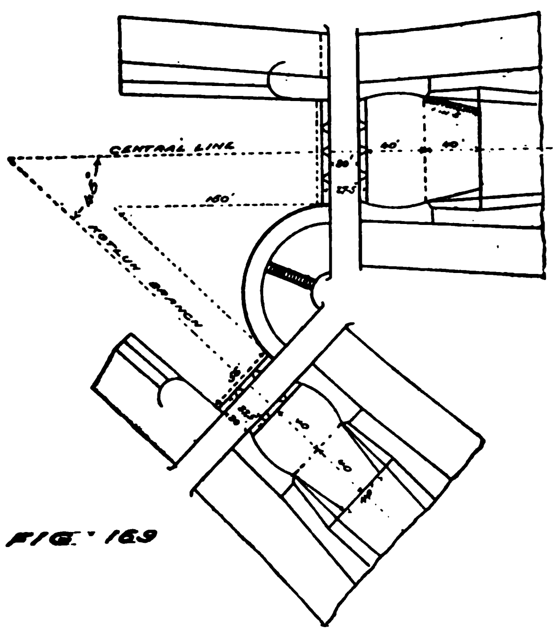


FIG. 169

Kotluh Branch Head at Suranah, Sutlej Canal.

the mean waterway of the channels below being 57.5 and 41.5 feet respectively, divided on the central into four bays, two of 14 feet each and two at the sides of 18 feet each; on the Kotluh branch, two at the sides of 18 feet each, with one central bay of 14 feet; the piers nearest the sides three feet thick, the central one two feet thick, built up to the same level as the tow-path.

* * * * * * * *

"One main object of the arrangement of the works, as shown on the plan, was to bring the bridges as close together as possible, so as partially to obviate the silting

[illegible] the upper channel [illegible]

[illegible] the fixing of the [illegible] in adjusting the supply, [illegible] to regulate altogether, if possible, by one bridge, leaving the passage through the central line quite free. The regulation at both heads will be effected by vertical sleepers, the needles already described, their lower ends resting in a groove in the flooring, confined above between two beams resting on the piers or side, retaining walls. This is an economical expedient, though, in some respects, not so efficient as the method with drop-gate, shown in Figure 92, still it will answer all the purposes of adjusting the supply. It has the advantage of dividing the entering stream into vertical films, by which the impact on the flooring will be diminished, and it can be worked by a couple of men."

Article 43. Escapes. Relief Gates—Waste Gates.

In order to provide for the control of the water in the [illegible] also called *Relief Gates* and *Waste Gates* [illegible] at certain intervals along the line of the [illegible] excess of water in the canal, and for which [illegible] may arise from a breach [illegible] at one of the [illegible] Extra- [illegible] banks and at [illegible] and there [illegible] necessity [illegible] the canal [illegible] shortest possible [illegible] without num-

dating or damaging the country through which it flows. The dimensions of the escape channel should be fixed so as to be large enough to discharge the maximum supply in the canal.

The location of an escape channel will be determined by the topography of the country, but, as a rule, connection from the canal can be made at intervals with some natural water-course.

An escape should be located above a heavy embankment, and above any part of the canal likely to be breached by floods.

Where possible to do so, they are, in India, provided at regular intervals along the line of the canal. Where they are taken off from the canal, a double regulating head should be built, as shown in Figure 170, one across the canal *AB* to prevent the water flowing down that

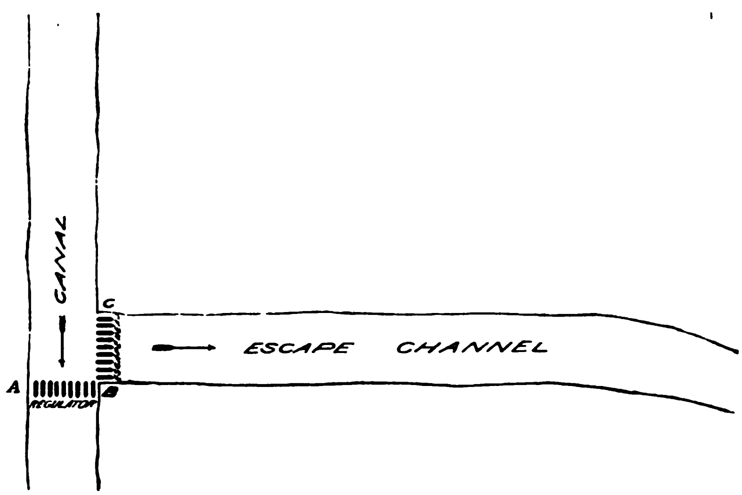


Fig. 170. Plan of Escape Head and Regulator.

way when the escape is in use, and the other across the escape head *BC* to prevent the water flowing down that way when the canal is in use.

Figure 166, page 221, shows the relative position of

an escape to a regulating bridge at an off-take of a branch canal on the Sone Canals, India.

To prevent artificial escape channels from being choked by brush they should be occasionally cleared, otherwise, when required for use, they may be found choked and prevent the discharge of the water, thus causing an inundation and the destruction of life and property.

On the Ganges Canal, India, escapes were provided every forty miles.

An escape near the head of a canal is sometimes used as a *scouring escape*. An instance of this is on the Agra Canal, India, where a scouring escape is placed one and a-half miles below the canal head. Its waterway is somewhat in excess of that of the canal head, and the object of this is to generate velocity enough in the first one and a-half miles of the canal, to stir up and carry away the silt deposited between the escape and the canal head.

In America, in order to save expense, waste gates are sometimes made in the sides of flumes, but this plan is liable to the objection that the falling water is likely to wash out the foundations and destroy the structure. A channel taken out in cutting, and connected with the water course, would be the safer plan; the bed and banks of the channel being protected from scour, by paving or some other method.

A few years since, the Naviglio Grande, the Muzza and Martesana Canals, in Italy, had no sort of regulating bridge across their heads, and the flood waters were allowed to enter the canal with their full force, finding an exit in a series of escape-sluices and weirs. The Naviglio Grande has a number of these sluices in the first few miles of its course, and two weirs running along its side of 300 feet and 65 feet in length, with their crests about three feet lower than the surface of the

canal full-water supply. These are blocked up by strong wooden fences, closed up tightly with bundles of fascines. The Martesana and Muzza Canals are also furnished with long over-fall weirs near their heads. That the sytem has gone on so long, among an intelligent people deeply interested in their irrigation, is sufficient proof that no very great harm can arise from it. The soil is so stiff and firm, that it is capable of resisting a heavy flood, and there are few masonry works near at hand to be damaged by it. There must, however, after a flood, be heavy deposits of gravel and silt in the canal channels.

Article 44. Depositing Basins—Silt Traps—Sand Boxes.

Depositing basins for large canals are fully described in Article 18, page 52, entitled *On Keeping Irrigation Canals Clear of Silt.*

Small channels taken from rivers carrying large quantities of sand or silt, sometimes have *Silt Traps* or *Sand Boxes* located at convenient points for clearing them out. These traps intercept the sand and silt carried by the water, and prevent the rapid silting up of the channel. These silt traps are flushed out, when required, with canal water. Care should be taken to locate them in such a position that the debris does not choke the outlets from the traps. In order to accomplish this the debris should be run into a channel that has a living stream, or that is scoured out occasionally by flood water. In consequence of neglecting this precaution in locating some of the silt traps on the Deyrah Dhoon Irrigation channels in India, their outlets got choked and, after some time, they became useless.

Mr. A. D. Foote, M. Am. Soc. C. E., fixed a trap and small gate for the purpose of intercepting and scouring out debris in a canal from the Boise River, Idaho. This

trap is a trench cut in the bottom of the canal, and running diagonally upward across it from the gate. In this trench all small stones and sediment that may be loosened from the high banks by spring thaws, will be caught, and on opening the gate they will be carried out of the canal by the rapid current through the opening.

On the Marseilles Canal, in the south of France, with a maximum capacity of 424 cubic feet per second, the water of which is used, not only for irrigation, but also for domestic use, settling basins were provided to rid the water of the sediment mechanically suspended, in order to render the water fit for domestic purposes. After several settling basins were silted up and rendered useless, one of them having a capacity of about 159,000,000 cubic feet, another large basin was constructed with the necessary works for flushing it out periodically. This basin has an area of fifty-seven acres, and its capacity is about 81,000,000 cubic feet. It is formed by constructing a masonry dam across a valley 654 feet in length, 72 feet in height, and 55¾ feet in width at the base. At the end of each year, when a deposit of about five feet of sediment has accumulated at the bottom, it will be flushed out into the river Durance at a low level.

Where the line of a canal passes through rolling ground, or skirts the bottom of low hills, a hollow in the ground, within a few miles of the head of the canal, is sometimes available and can be utilized for the deposition of silt. Sometimes a few low and cheap dams have to be built in the lower depressions.

The silt-laden water enters the reservoir at its upper end. Its velocity is then checked, and it deposits its load of gravel, sand and slime, and after passing through the reservoir, it again enters the canal at its lower end, comparatively clear water.

No doubt it is only a matter of time for such a basin to fill up and become useless for its intended purpose, but, as the following instance will prove, a useful depositing reservoir can, with due forethought, be made at a small expense.

The Wutchumna Canal, in Tulare County, California, is taken from the right bank of the Kaweah River, at a point where this river sometimes, when the water is most required, carries large quantities of sand and silt. The clearance of this sand and silt at the close of the irrigation season, from other canals in the same district, entails heavy annual expense.

When locating the Wutchumna Canal, Mr. Stephen Barton, C. E., with happy forethought, carried it through a hollow in the ground with the intention of converting the hollow into a depositing basin. This he accomplished successfully, and the writer is not aware of any depositing basin in existence, of the same capacity as the Wutchumna reservoir, that is so well adapted to the duty it has to perform.

This reservoir is situated about seven miles from the headworks of the canal, and the velocity of the canal, through this seven miles, is sufficient to prevent the deposition of sand and gravel until it enters the reservoir.

Mr. W. H. Davenport, C. E., the present Superintendent of the Wutchumna Canal, has lately sent the writer the following additional information on this subject:

"When the reservoir is at what we call low water, just now, its area is 61 acres, with an average depth of 3 feet. What we call a full reservoir is 154 acres in area, and has a depth of 7 feet above low water. The discharge of the Wutchumna Canal is 208 cubic feet per second.

"There is at present an average depth of 1.25 feet of deposit over the lower water area of 61 acres. Where the ditch enters the reservoir I find a *bar* of sand and

gravel, which the high grade of the canal has carried. This bar I estimate to have an area of 20 acres, and a depth of 3 feet.

"The Wutchumna Canal has been in continual use for 10 years, drawing its supply every day without interruption. I think I can safely say that, the reservoir will be useful for a silt deposit for the next 100 years. The conditions are such that the reservoir can be made 6 feet deeper."

Article 45. Tunnels.

There are occasions, as explained further on, when a tunnel can be adopted with advantage, but they are seldom used when the supply required is over 2,000 cubic feet per second. There are no tunnels on any of the large irrigation canals in India that discharge over 2,000 cubic feet per second.

The High Level Canal in Colorado, with a discharge of 1,184 cubic feet per second, has a tunnel at its head 600 feet in length. It is 20 feet wide and 12 feet high, with a grade of 1 in 1,000.

The Merced Canal in California, with a discharge of 3,400 cubic feet per second, has a tunnel 1,600 feet in length, through solid rock, and another tunnel 2,000 feet in length, through ground so unstable, that it was necessary to timber its whole length, a work which required over 1,000,000 feet, board measure, of redwood. In India, timber in a similar position, would, in a few years, be destroyed by the white ants.

The Henares Canal in Spain, with a discharge of 177 cubic feet per second, has a tunnel 9,513 feet in length. The tunnel is lined throughout with brick. It has a semi-circular arch on top, and an inverted arch on the bottom. Its height at the center is 11.2 feet, its width at springing of invert 7.2 and its grade 1 in 3,067.

In Madras, India, a tunnel is to be constructed to convey the waters of the Periar River into the Viga Valley for irrigation. This tunnel is in rock 6,650 feet in length. Its cross-sectional area is 80 square feet and it has a slope or grade of 1 in 75.

Under certain conditions a tunnel,* when in sound rock, is preferable to an open channel for conveying water. The conditions are, that no water is required to be drawn off this part of the line, and that a heavy grade can be given. By sound rock is meant rock not subject to percolation, to any appreciable extent, that will stand the high velocity without injury by erosion, and also that will not require lining for its sides or arching for its roof. When, in addition, a steep grade can be obtained, a high velocity can be given to the water, and the cross-sectional area and consequent expense reduced.

In such a tunnel, the loss of water by evaporation and percolation, and the expense of maintenance are at a minimum. It has several advantages over the open channel in steep, side-hill ground. Its sides and bed are impervious to water, and it is covered from the sunlight. It shortens the line, there is no compensation to be paid for land, and it does not interfere with or cross the drainage of the country on the surface. Should it be required, at any future time to increase the carrying capacity of the canal, the discharge of the tunnel can be increased without, however, increasing its dimensions. See *Flow of Water*.

All that will be necessary is to fill all the hollows between the projecting ends of the rocky bed and sides with good cement concrete, and after this to give a coat of good plaster to the surfaces in contact with the water and make them smooth. Although the section will be

* Report on the proposed Works of the Tulare Irrigation District by P. J. Flynn, C. E.

diminished, still, the velocity and consequent discharge will be doubled.

Let us assume the loss of water in a certain length of open channel at six per cent. of the total flow. If by adopting a tunnel line, the loss of water is only one per cent., it is evident that it would pay to expend the value of five per cent. of the water on the tunnel line above that on the open channel.

Another argument in favor of the tunnel is that the amount saved yearly in maintenance capitalized could be expended on the tunnel over that upon the open channel, in order to give a fair comparison with the latter. See *Flow of Water*, page 52.

On the Marseilles Canal, in France, there are, in all, ten miles of tunnelling, the mean velocity through them being nearly 5 feet per second. The maximum discharge of this canal is 424 cubic feet per second.

On the Verdon Canal, in France, the number of the tunnels is seventy-nine, of a total length of twelve and a-half miles, the three most important of which are respectively about three and one-fourth, two and five-eighths and one and seven-eighths miles in length. The capacity of the main canal is 212 cubic feet per second, and it has a sectional area of 113 square feet.

Tunnels are employed in several instances in Southern California, to *develop* water. Where there is a water-bearing strata a tunnel is driven, and in several instances, sufficient water has been developed to make the money expended a good paying investment, and by the use of this water for irrigation, land has been raised in value from \$5 to \$500 per acre.

A good example of this kind of work is the San Antonio Tunnel, which is being constructed at Ontario, Southern California, by F. E. Trask, Chief Engineer of the Ontario Land Improvement Co., who has supplied the following account of the work:

SAN ANTONIO TUNNEL.

At an early date the founders of Ontario concluded they would need a larger supply of water than the one-half flow of San Antonio Creek—which gave them 365 miner's inches—and it was decided to tunnel for the underflow of this creek, at the point where it enters the San Bernardino Valley. Land controlling the mouth of the cañon having been secured, the work of driving the tunnel began in the early part of 1883. The objective point of the tunnel was the lowest point of *bed rock* in the cañon about one-half mile from its mouth. Here it was estimated that from eighty to one hundred feet of gravel, bowlders, etc., laid above bed rock. It was decided to start the tunnel about 3,000 feet south of the objective point and run on a grade of one-half inch per rod—with a cross-section of twenty-eight square feet. The alignment and grade have not been strictly adhered to, although no serious changes have been introduced. The first two thousand seven hundred feet were driven through the rock and gravel formation of the cañon, and required lining, which was as follows: the bents were of 8″x8″ redwood and spaced 4 feet center to center—the bed pieces were 2″x8″ and the lagging 2″. The clear dimensions were, height 5′ 6″—top width 2′—bottom width 3′ 6″.

The above portion of the tunnel has been lined in the following manner: slabs of concrete, four inches thick, were laid in hydraulic cement over the entire bottom. Between the bents and on these concrete slabs for a foundation, the side walls eight inches thick were carried up of concrete blocks (rock was used in some portions of this section), to a height of 4′ 2″ on which the arch was turned. The arch was composed of two segments, with a tongue and groove joint at the center. These walls and arches were laid in cement, care being

taken to make water-tight joints. The only deviation from this was at points where veins of water were intercepted, there, rectangular openings, of sufficient size and number, were left to admit the water at a height of two feet above the bottom of the tunnel. The accompanying section shows both the method of timbering and lining.

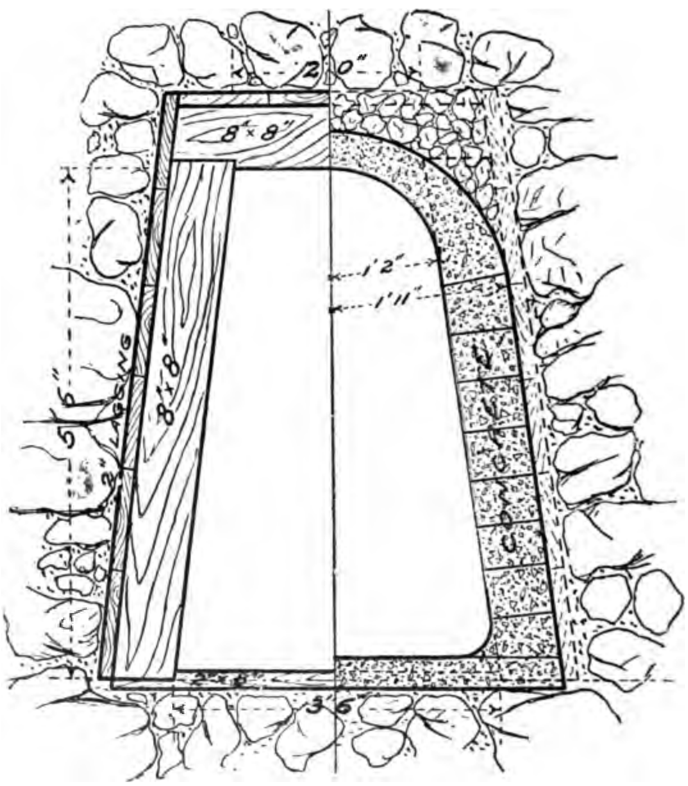


Fig. 171. Cross-Section of San Antonio Tunnel.

Bed-rock was reached at two thousand seven hundred feet, and up to January, 1891, the tunnel had penetrated six hundred feet into bed rock, making a total length of tunnel of 3,300 feet. At this time the heading was con-

sidered to be beyond the lowest point in bed rock, and it became necessary to investigate the material above the roof of the tunnel. For this purpose a *diamond drill* plant was procured, and about four months work was required for defining the surface of bed rock. From the data thus obtained, it was found that *three low points* in bed rock existed—one about fifty feet from the point where bed rock was first struck; another 340 feet; while a third was found to be about 200 feet beyond the heading of the tunnel. Up to the time bed rock was struck, the minimum flow of the tunnel has been about fifty miner's inches. On September 15, 1891, there were 137 inches; and at the present writing (October, 1891), about 300 inches have been developed. As yet the work of development above bed rock has hardly begun, and from one to two years work will be required to complete the proposed plans, when it is believed 1,000 inches will have been developed.

In general terms the proposed method of development consists of a complete network of supplementary tunnels and drifts above bed rock, and on the *up stream* side of the main tunnel, which will be connected by means of shafts to the main tunnel some twenty feet below. On bed rock on the *down stream* side of the main tunnel will be built a submerged dam of sufficient height to intercept the summer underflow.

Seven shafts have been used in the entire length of tunnel, and increase in depth from No. 1, 20 feet, to No. 7, 104 feet. They are unevenly spaced, the greatest run being 600 feet.

Quicksand was encountered at several places and gave much trouble. Cost: The cost of driving the first twenty-seven hundred feet, including temporary wooden lining, varies from $2.50 to $20 per lineal foot. The contract for concrete lining was $2.50 per lineal foot.

.. ..ed tunnel (2,700 feet), as
... was about $50,000. The
.. rock was $8 per lineal foot.
..ing of any kind is required.
..ing, above bed rock, has not
... ustify a statement of cost, at

... 40. Retaining Walls.

... mulae have, from time to time,
... the thickness of retaining walls,
...erably in the results obtained by
... ..ws, of May 24th, 1890, states with
... hickness of wall at any height:—
... pet formula which we want to air
... short and simple: 'three-seventh
... row in some odd inches for luck,'
... o be more strictly and more truly
... one of a number of much more com-
... 'he before us. It is certainly fool-
... ..ing walls much thinner than this
... ..der any conditions. While expe-
... ..y any well-built wall, proportioned in
... 's pretty sure to stand."
... practical engineering in the above
...ng articles discussing the pressure of
...wall, under various conditions. The
... above formula are probably intended
... ..ents of the materials composing the
...ferent specific gravity. Another rule is
... height in feet plus one is equal to the
... rule gives almost the same result as that
... *Engineering News.*
... ..n course of retaining walls has its width
... ..d the thickness of the wall, by a series of

steps in front, two only are shown in Figure 172. The objects of this are at once to distribute the pressure over a greater area than that of any bed joint in the body of the wall, and to diffuse that pressure more equally by bringing the center of resistance nearer to the middle of the base than it is in the body of the wall.*

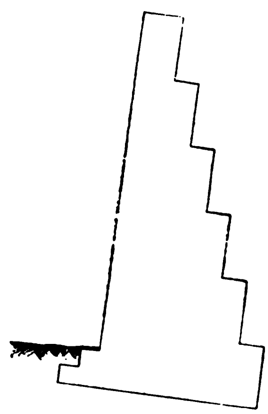


Fig. 172. Cross-Section of Retaining Wall.

The body of the wall may be either entirely of brick, or of ashlar, backed with brick or with rubble, or of block-in-course backed with rubble, or of coursed rubble, built with mortar, or built dry. As the pressure at each bed-joint is concentrated towards the face of the wall, those combinations of masonry in which the larger and more regular stones form the face, and sustain the greater part of the pressure, and are backed with an inferior kind of masonry, whose use is chiefly to give stability by its weight, are well suited for retaining walls, special care being taken that the back and face are well tied together by long headers, and that the beds of the facing stones extend well into the wall.

Along the base and in front of the retaining wall

*Rankine's Engineering.

there should run a drain. In order to let the water escape from behind the wall, it should have small upright oblong openings through it called "weeping holes," which are usually two or three inches broad, and of the depth of a course of masonry, and are distributed at regular distances, an ordinary proportion being one weeping hole to every four square yards of face wall.

The back of the retaining wall should be made rough, in order to resist any tendency of the earth to slide upon it. This object is promoted by building up the back in steps, as exemplified in Figure 172.

W[illegible] at the back of the wall is clean [illegible] can pass through it readily [illegible] necessary [illegible]

[illegible]

[illegible] Irrigation and Navigation Canals.

[illegible]

in proportion to the increase in velocity of the water, and, consequently, all the works, such as headworks, embankments, cuttings, bridges, flumes, falls or drops, etc., can be diminished in size and expense. In addition, locks to pass the falls would be required for navigation.

Mean velocities exceeding 4 feet per second cause waves, which injure the banks in the greater number of canals, especially in sandy loam.

An irrigating canal requires at least, for average ground, a velocity of 2½ feet per second. It follows, therefore, that when forcing its way against the current at the rate of 4 feet per second the boat is actually making headway only at the rate of 1½ feet per second, and any attempt at quicker velocities would injure the banks, so that, irrespective of the loss of power, the banks could not stand if there was quick navigation. It, therefore, appears evident that for economical working and the safety of the banks, an almost still water canal is required.

Indian experience has fixed about 1½ feet per second as the maximum velocity which ought to be allowed in a navigable canal. The small slope would increase the number of falls required to overcome the greater surface slope of the country, and in addition, the greater cost of all the other works would make the cost of a navigable canal almost double that of the channel required for irrigation alone.

Again, in a navigable channel, a certain minimum depth and width, for the passage of canal boats, must be allowed everywhere; and the amount of water required for this minimum must be allowed over and above the quantity required for irrigation. This has been referred to in Article 5, page 11, entitled *Quantity of Water Required for Irrigation*.

be considerably lessened by the diminution of the excavation for berms or tow-paths, and the reduction of the width of, and headway under, the bridges, to that necessary for the mere passage of water supply. The latest information on the subject of Navigable Canals, in India, is strongly in support of the above.

In the Revenue Report of the Irrigation Department of the Punjab, India, for 1889–90, it is stated with reference to the Sirhind, or Sutlej Canal, that: "On this, as on the other irrigation canals of Upper India, the cost of providing navigation is not likely to prove remunerative." This is conclusive.

Article 48. Survey.

The same rules which govern the survey of a railroad line are also to be observed in the survey of a canal line. There are, however, a few points which it is well to refer to here.

The level of the floor of the head gate of the canal is a good datum for zero for levels, and the face of the up-stream head wall of the head gate is suitable for the zero of longitudinal measurement for the central channel of the canal, and the same plan can be adopted on their respective regulating head gates in fixing the same points for the branches and laterals.

Correct levels are of primary importance in canal lines, and it is advisable to level twice over the same stations with the same instrument, the second levels being carried in the reversed direction to the first. In a canal carrying over 1,000 cubic feet of water per second, a few inches more or less in a mile will make a serious difference in the velocity.

It is advisable to have frequent *bench marks*, and on permanent objects where possible, and all canal, road, railroad and other bench marks should be connected

with the line of levels. A bench mark should be established close to each heavy cut and fill, crossings of all rivers, canals, bridges, aqueducts and other works on the line of canal.

Where possible to do so, without extra expense, sharp curves are to be avoided. In India, in the plains, flat curves are adopted varying from 5,000 to 15,000 feet in radius. In the Isabella II Canal, in Spain, a recent work, with a discharge of only 89 cubic feet per second, the maximum radius was fixed at 492 feet and the minimum at 328 feet.

Cross-sections should be made of all ravines and water-courses crossing the line of canal, and cross-sections, at right angles to the axis of the stream, should be taken in all channels subject to flooding. The cross-sections should show the surface of the water at the date of observation, and the ordinary and highest flood marks.

The waterway of all bridges and culverts and the levels of their floors, if any, and the lowest part of the superstructure should be noted.

The nature of the ground should be noted, and enquiries should be made as to whether the country is flooded, and as to whether there is any alkali land passed over by the canal line.

In India, in a generally level country, the following plan is adopted preliminary to the survey for a main canal. Cross-sections are taken at intervals, perpendicular to the supposed water-shed. For the general alignment of the main channels between two large rivers, the interval should not exceed ten miles. For the actual location and for the minor channels, the interval probably should not exceed five miles, or possibly less. The cross-sections should be connected by longitudinal lines at their extremities, to test the accuracy of

the work. These levels being platted on a map on a large scale, the line of canal can be laid down *approximately* on the map as a preliminary to the location. The levels will also show the general directions of branches and laterals, and also the natural drainage lines of the country.

If the levels of the water-shed admit of it, the nearer the canal line approaches to it the better, as the interference with surface drainage of the country will then be the least possible. Having determined the lines of the main canal and its branches the next thing to do is to locate the distributaries.

In order to deliver the water under the most favorable conditions, it is clear that the irrigating channels must everywhere follow the water-sheds of the country drainage.

An almost perfect arrangement of distributaries is exemplified in Figure 173, taken from a paper by Mr. H. M. Wilson, M. Am. Soc. C. E.* This arrangement shows the distributaries following the water-shed lines of the country. It is seldom that such a complete distributary system can be located.

The first step then, is to ascertain how many water-shed lines exist, their extent and relative situations. This knowledge can only be obtained from a careful survey of the country it is designed to irrigate, care being taken to delineate on the map the course of all rivers, streams, roads, railroads, canals, etc., and the position of all hollows, swamps and the other salient points of the topography of the country. To each water-shed should be assigned a separate channel of capacity apportioned to the duty it has to perform, the two bounding streams or drainage channels being

*Irrigation in India in Transactions of the American Society of Civil Engineers. Vol. XXIII.

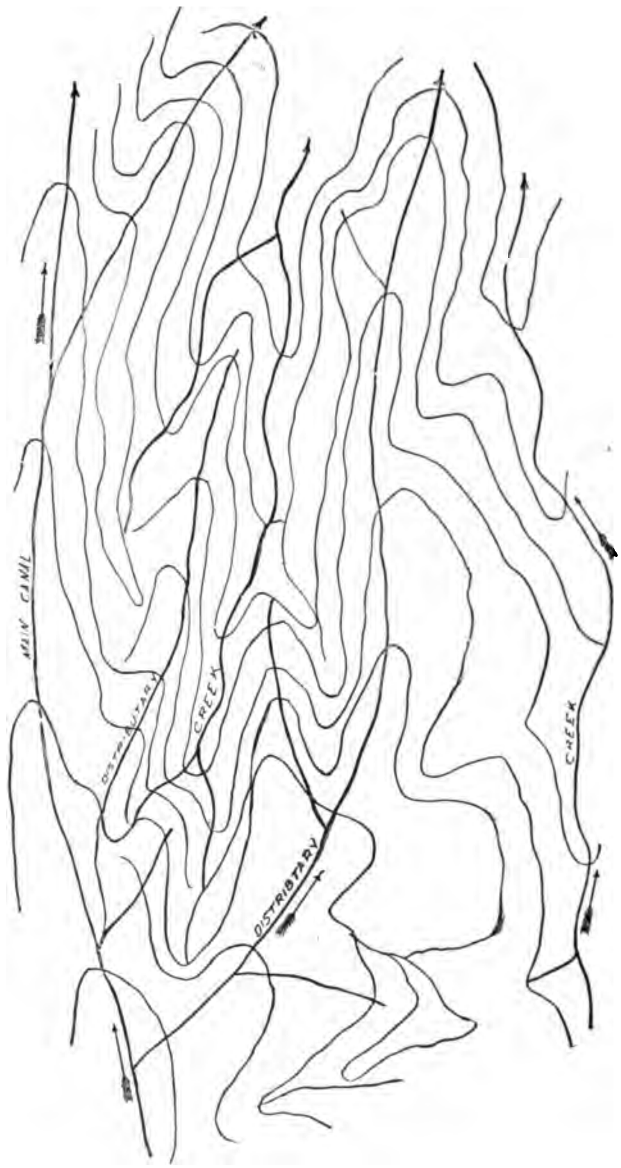


Fig. 173. Drainage Map Showing Arrangement of Distributaries.

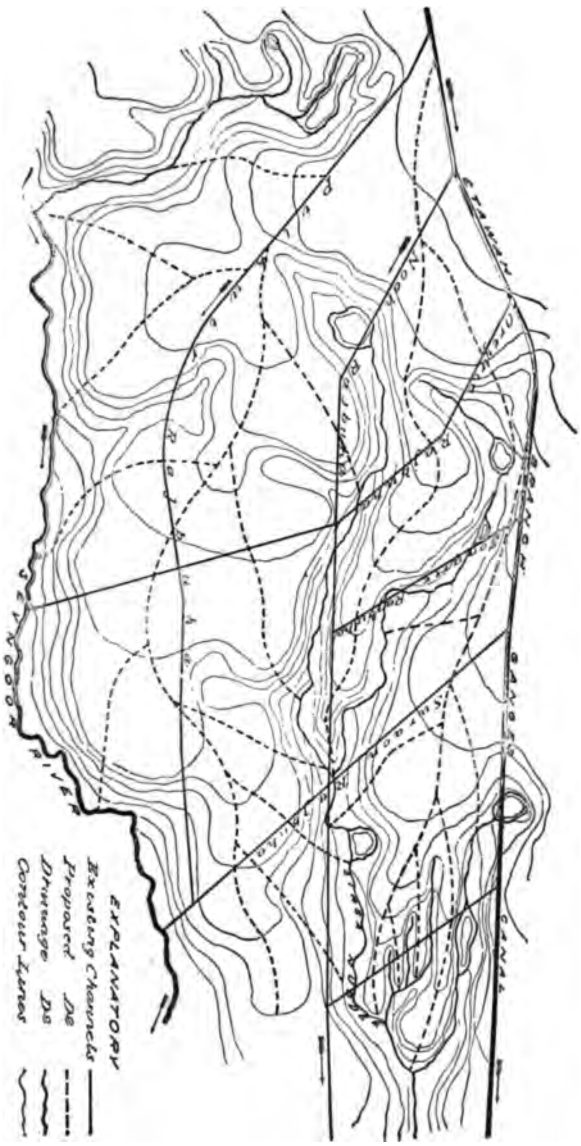


Fig. 174. Plan Showing Arrangement of Distributaries.

considered in this system as the limits to which irrigation from any single line should be carried. This is very plainly shown in Figure 173.* Figure 174 shows a defective location of a distribution system and the proposed improvements. The difference between the location of the laterals on the two Figures 173 and 174 will be apparent on inspection. On the former the channels are kept on the water-shed lines all through, but on the latter the original channels depart so much from the water-shed that large tracts of land cannot be irrigated.

Having then traced out, as above stated, on the drainage survey map the general course of the proposed channels, it is necessary to run a series of cross-levels in order to fix the exact position of the water-shed. With the aid of the information thus obtained, the engineer will be enabled to locate the distributary to the best possible advantage.

Some American engineers may think that too much time and labor is given, by the above method, but the experience of Indian engineers, on thousands of miles of badly located distributaries, proves that too much thought and care cannot be given to the location of these channels.

For the more complete and efficient distribution of the water, minor distributaries should be taken out from the main distributaries where they may be most required; but the engineer should in a measure be guided by the nature of the ground and the character of the soil. As in the case of larger works he should endeavor to secure a command of level for the purpose of affording every facility for irrigation; he should avoid as far as possible crossing minor drainages or stumbling into hollows, by which his object may in any measure be de-

*Professional Papers on Indian Engineering. Vol. IV. First Series. Captain W. Jeffreys, R. E.

feated; he should banish from his mind any idea he may entertain of the relative unimportance of this class of works; for he may be assured that nothing tends so directly to an economical distribution of the water as a carefully constructed system of minor distributaries.

In the system advocated above, the capacity of an irrigating channel should everywhere be exactly apportioned to the duty it has to perform, the section decreasing as the line advances until it loses itself in a small water-course. See Article 8, page 20.

The level of the bed of the distributary should be fixed rather with reference to the full supply level of the canal, than to the level of the canal bed, chiefly because it is an object to keep the bed of the distributary at a sufficiently high level to admit of surface irrigation on its whole line as far as possible. Moreover, the nearer to the surface that water is taken off by a distributary head, the less will be the silt which enters the distributary, and the less the annual labor of clearing the bed. The bed of a distributary will, therefore, generally be from 1 to 3 feet higher than that of the main canal.

When the Eastern Jumna Canal, India, was laid out, the main line was constructed by the engineers, the distribution channels being left to be made entirely by the cultivators. That led to such great evils, that when the Ganges Canal was made, the main distribution channels were laid out and constructed by the Government engineers; but the minor ones were still left to the cultivators to make; and on the Agra Canal a complete system of distributaries was carried out as an integral part of the scheme.

Mr. Forrest* had charge for six years of one of the divisions of the Ganges Canal. It was a tail division,

* Mr. R. E. Forrest, M. I. C. E., in Transactions of the Institution of Civil Engineers. Vol. LXXIII.

where the supply of water was not great, while the demand was large. The engineers had to make water go as far as possible. When he first went there the waste of water was enormous. The cultivators had taken their channels in all sorts of wrong places, down roads and hollows, and across waste lands, and waste water was lying about everywhere. One great cause of loss was this: the country was studded with barren plains, and when a main distributary ran across one of them, the good land on either side was irrigated by means of little channels across the plain, as shown by the dotted lines in Figure 175, some of them over a mile

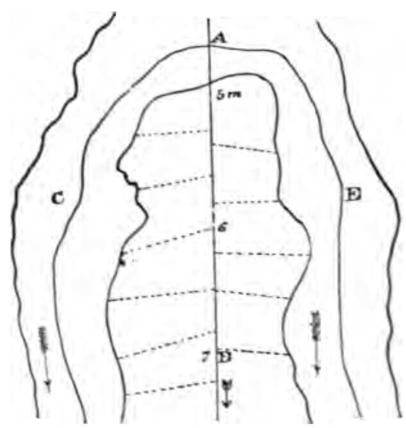


Fig. 175. Plan Showing Arrangement of Distributaries.

long. There was great loss in having so many channels; and, as the banks were made of the silty soil of the plains, and badly made, they were always failing and flooding the plain, which no one minded, as the land was barren. For these channels were substituted properly laid out channels, A C, A E, through the middle of the good land, which, having banks made of good earth, did not break down, and if they did good lands were flooded, so that the canal establishment and

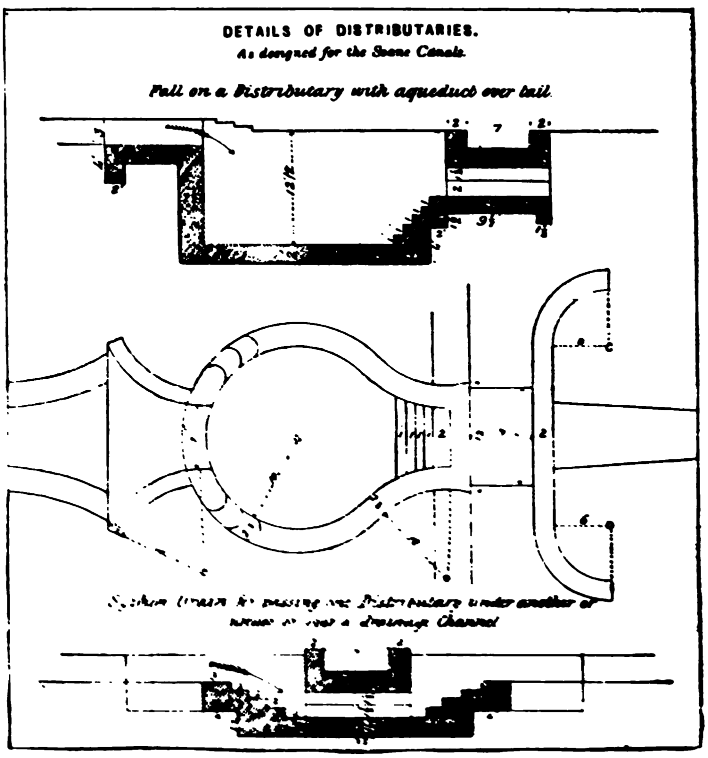
the cultivators had to take pains not to let them fail. The effect of the change was wonderful. Mr. Forrest had two channels, one 40, the other 50 miles long, running through land of that character; and whereas he had previously not been able to get the water half way down them, he then got it down to the very tail. That led him on to making as many of these minor distribution channels as he could. Each of these little watercourses was dealt with exactly as if it had been a big canal. Careful surveys were made and levels taken for it. The line was located, and the longitudinal section and cross-sections carefully fixed. Badly adjusted cross-sections caused a great loss of water. People laughed at so much pains being taken with such small channels, but the labor was not thrown away. That division became one of the best paying ones on the canal, and some of these channels gave a duty of 400 acres per cubic foot per second.

Thus, then, the first thing was to make the distribution channels properly, and the next thing was to work them properly. The water should be moved about and distributed by a careful system of rotation. It was better to move the water in as large volumes as possible. By a good system of rotation, it might be possible to remedy the loss of duty from the water not being used at night; the water could be run on at night to the more distant points. By a system of rotation, the evils of super-saturation could be lessened. The water was made to run through a tract only when it was wanted, and for so long as it was wanted. In some of the Ganges canal channels, the water ran only for a single day each fortnight. The water should be completely drawn from every tract in which it was not in active and immediate demand.

Article 49. Distributaries—Laterals—Rajbuhas.

These channels are also called Distribution Channels, Primary Channels, Ditches, etc., and they derive their supply from the Main Canal.

These channels are, in every respect, a counterpart of the main canal, and require the same class of works, though on a smaller scale, as the main canal.



Figs. 176, 177, 178.

Figure 176 illustrates a section of a fall on a distributary with a small aqueduct over its tail to carry another small distributary, and Figure 177 is a plan of the same.

Figure 178 is a section of a syphon drain for passing one distributary under another, or under a drainage channel.

The design, location, construction and maintenance of distributaries should be as carefully carried out as that of the main canal, for on all these details the economical use of the water will chiefly depend.

Those people acquainted with irrigation centers in America, are aware that proper attention to the minor channels of an irrigation system is very seldom given in this country.

In India, on the older canals, irrigation was carried on from the main channel itself, that is the small irrigation outlets were fixed in the canal banks. On account of the leakage along the outside of these pipes, frequent breaches of the banks took place. During years of

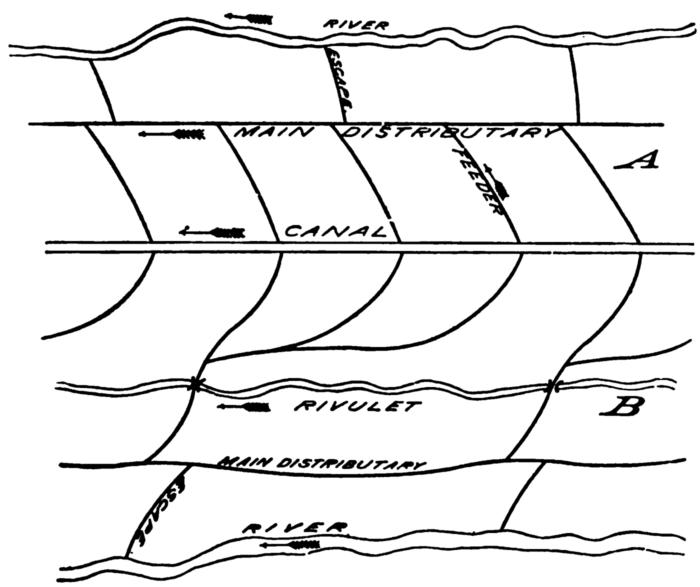


Fig. 179. Plan of Distribution System.

drought the villagers cut the banks and attributed the breach to some other cause. The loss of water resulting from these practices and several other causes, were found to be so great, that the distribution (rajbuha) system is

now generally adopted. In this system all pipes or tubes for the direct irrigation of land, must be taken from the lateral, and not from the main canal.

Figure 179 is intended to illustrate the system of locating the distribution channels in use in Northern India.* In this system, as remarked by Sir Proby Cautley, the greatest canal engineer that ever lived, we may consider the canal as answering to the reservoir or supply channel, in the water supply of towns, the distributaries as the mains, and the village water-courses as the service channels. The village water-courses are not shown in any of the diagrams in this article.

A and *B* show the methods ordinarily used there where the slope of the country is so flat as seldom to admit of the waters of the distributary being returned to the canal. In order to have the same velocity as the main canal a distributary must have a greater grade, and where the slope of the canal is parallel to the surface of the country, it is evident that after a channel with a greater grade than the canal has left the latter, it cannot again return its water into it. In order to have the same velocity, the grades, required for a canal and distributary, are in the inverse proportion to their hydraulic mean depths.

Where the slope of the country is greater than that of the distributary, Figure 180, *C* and *D*, show different methods by which the tail water of the distributary is returned to the canal. *C*, in the diagram, gives an example how this may be done in a case where the canal is too far in soil to afford water at a proper level to irrigate close to its banks. After leaving the canal in cutting at [illegible], etc., the distributary does not gain sufficiently on the grade of the country to be able to give surface

*Sone Canal Project by Col. C. H. Dickens.

elevation until it arrives at b^1, b^2, etc., passing there, over a syphon or fall conveying the returning upper distributary, which from loss of level in the crossing does not irrigate again till it comes to d^1, d^2, etc., whence it passes over the distributary next but one below it, and irrigates the land close to the bank, before it returns by a drop into the canal. An arrangement of this kind could only be effected with a very good fall of country.

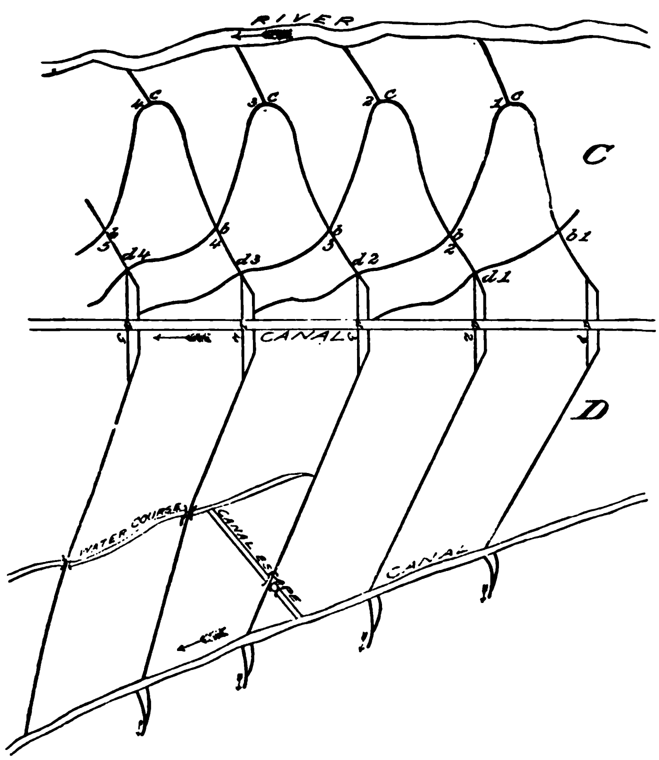


Fig. 180. Plan of Distribution System.

In D, in diagram, the tail water from the upper canal is intercepted and utilized by the canal located on a lower level.

The above illustrations are given to show what has

been done in Northern India, where irrigation has been carried on from time immemorial, and where the British Government have developed it to an extraordinary extent.

In locating laterals an engineer must be careful not to attempt to be too systematic, but to be guided by his own ingenuity and the nature of the ground in each case. In the American plains, distributaries are often carried along fence lines, which form sides of either rectangular or square tracts of land.

Fig. 181.

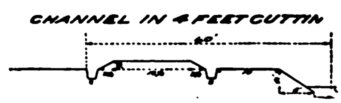


Fig. 182.

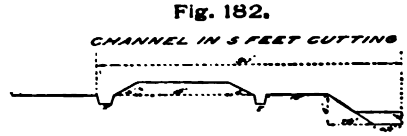


Fig. 183.

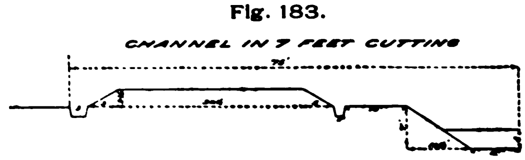


Fig. 184.

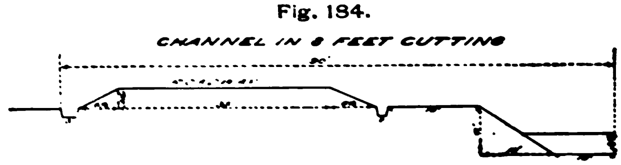
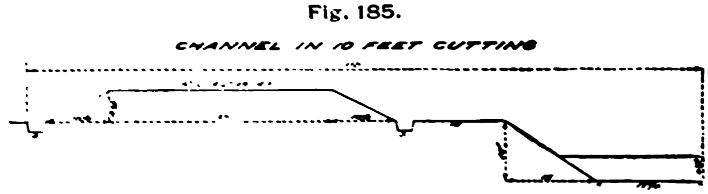


Fig. 185.



Figures 181 to 185 exemplify distributaries *in cut* de-

signed for the Sone Canals, India.* Only half the bed width is shown. These illustrations are given, not only as good specimens of design, but also to show the care that is taken in India, with even the minutest details of design.

Distributaries may be cleared of silt whenever the water is least required. One, or at most two clearances a year are enough for a well designed distributary. The floorings of all bridges and other masonry works, built over them, will, of course, have been carefully laid down to the proper levels, and will give so many permanent bench-marks for restoring the correct level of the beds; besides which, stakes or masonry bench-marks should be fixed at intervals, not exceeding a furlong.

Major Brownlow states, that the greater the amount discharged by a distributary, the smaller will be the proportion of cost of maintenance to revenue derived. This is evident, when we consider that, other things being equal, a channel having a bed width of 12 feet, and side slopes of 1 to 1, discharges almost double the volume discharged by two, each having a bed width of 6 feet, while the cost of patrolling and repairs to banks of the 12 feet channel will be about half of that on the two 6 feet channels. When, however, the channel silts up as illustrated in Figure 7, page 19, and the side slopes average ½ horizontal to 1 vertical, the 12 feet channel discharges more than two 6 feet channels. The transporting power of large volumes of water being also greater than small volumes, the deposit of silt in the 12 feet channel will be less in proportion to the discharge than in the two 6 feet channels, thus doing away with the necessity of frequent clearances required in the latter.

*The Sone Canal Project by Col. C. H. Dickens.

The following table based on Bazin's formula (37) *Flow of Water*, for channels in earth, is proof of what has been stated:—

TABLE 17. Giving velocity in feet per second, and discharge in cubic feet per second, of channels with different bed widths, but all other things being equal, based on Bazin's formula for earthen channels.

Bed width in feet.	Depth in feet.	Grade	Side slopes.	Velocity in feet per sec.	Discharge in cubic feet per second.	Side slopes.	Velocity in feet per sec.	Discharge in cubic feet per second.
3	3	1 in 2500	1 to 1	1.43	25.65	½ to 1	1.39	17.34
6	3	1 in 2500	1 to 1	1.65	44.60	½ to 1	1.58	35.59
9	3	1 in 2500	1 to 1	1.80	64.66	½ to 1	1.76	55.34
12	3	1 in 2500	1 to 1	1.90	85.28	½ to 1	1.87	75.83
15	3	1 in 2500	1 to 1	1.97	106.26	½ to 1	1.95	96.74
18	3	1 in 2500	1 to 1	2.02	127.46	½ to 1	2.02	117.90

By adopting large distributaries the actual amount of clearances during the year is also diminished, for a great portion of the silt which would be rapidly deposited at the head of a small line, is carried along and dropped into the water-courses branching off from a large one.

In Northern India distributaries are of various sizes discharging from 4 to 200 cubic feet per second, but experience seems to prove, that irrigation may be safely and most profitably carried on from channels 18 feet wide at bottom, with side slopes of 1 to 1, the depth of water being from 3½ to 4 feet, provided that the depth be kept at least 2 feet below soil for the first ten miles of its course, and that no outlets be allowed in subsequent embanked portions of the line.

On the Eastern Jumna Canal during 1858–59 and 1859–60, the revenue from all distributaries of 12 feet

head water-way and upwards, amounted to $64,809, while the expenditure on their maintenance was $8,019 or .123 of the revenue. The revenue from all distributaries below 12 feet water-way at the head was $133,-524, and the cost of maintenance $28,289, or .223 of the revenue, being very nearly double the proportion in the first case.

The head mentioned is the *width* of water-way of the regulator at the head of the distributary. For example, if a regulator at the head of a distributary has *one* clear opening of 12 feet between the abutments, that is called a 12-foot head, but if there is a pier in the center making two clear openings of six feet in width each, this regulator would also have a 12-foot head.

The economy of water on the large channels is equally marked, for, during the above-named two years, the revenue was, from:—

Seven distributaries of 12 feet head water-way and upwards, $64,809;

Forty-nine distributaries of 6 feet head water-way and upwards, $108,216;

Twenty-nine distributaries of 3 feet head water-way and upwards, $25,308;

Giving an average revenue per annum of:—

$4,629 from a distributary of 12 feet head water-way.

$1,104 from a distributary of 6 feet head water-way.

$436 from a distributary of 3 feet head water-way.

Measurements made gave 90, 32 and 22 cubic feet per second as the relative discharges from 12 feet, 6 feet and 3 feet heads on this canal; from which we have as the relative values of a cubic foot of water per annum:—

$51 on a 12 foot distributary.

$35 on a 6 foot distributary.

$20 on a 3 foot distributary.

The increased action of absorption and evaporation

over the greater area covered by water of the smaller channels, accounts for the difference above shown.

The depth of water in distributaries should seldom exceed 4 feet: but in carrying out a new line of irrigation, we should aim at keeping the surface of water at about 1 to 1½ feet above the general surface of country, so as to secure irrigation by the natural flow of water. Under these conditions, breaches in the banks need never be feared, with ordinary care in their construction and maintenance. This object, however, is to be kept in due subordination to the primary *desiderata* of a reasonable longitudinal slope, and an alignment following the watershed of the country.

Where the existing supply on a distributary becomes insufficient for the demand, it will be, in the end, found more economical to increase the discharge by widening the original channel for a suitable distance, than to do so by carrying the required additional volume down from a second head, as used to be often done. Against the latter course, all the arguments before adduced hold good, while the back-water from the head which is running the strongest, is sure to check the velocity of the water in the other, and so immensely accelerate the deposit of silt.*

The Roorkee Treatise states, that the system of raising water to the level of the country, where it runs below the surface of the soil, by *stop dams* or *planks*, introduced into grooves constructed for that purpose, cannot be too strongly condemned. These convert what should be a freely flowing stream, into a series of stagnant and unwholesome pools, encourage the growth of weeds and the deposit of silt, and are in every way objectionable. Besides, with a reasonable slope in the surface of the country, it will generally be found that, for every acre of

* Roorkee Treatise on Civil Engineering.

land thus secured, ten can be obtained further on by the natural flow of water. Be this, however, possible or not, it is decidedly better to resort to any other means of raising the water to the level of the country than the above wasteful and unhealthy expedient.

CROSS-SECTIONS OF DISTRIBUTARIES.

IN EXCAVATION

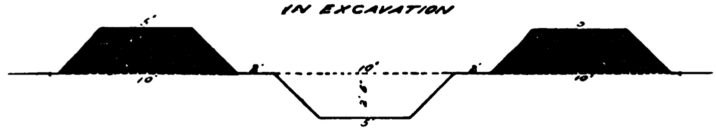


FIG. 186

IN EMBANKMENT

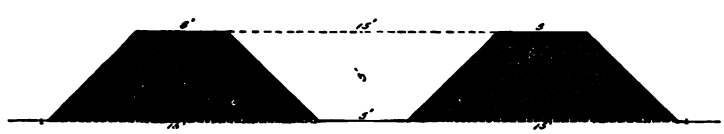


FIG: 187

IN EXCAVATION

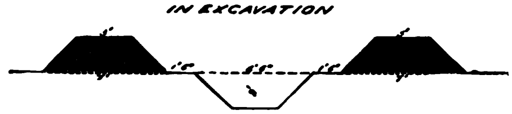


FIG: 188

IN EMBANKMENT

FIG: 189

The writer has seen, in California, land irrigated by the use of stop planks that, without their use, could not have been irrigated. No bad results whatever followed

from raising the water. As a rule, there is no necessity to keep the stop planks in more than twelve hours, and in this short time little if any damage can result. It is simply a question of utilizing so many acres of land by raising the water for about twelve hours at certain intervals of time.

Figures 146 to 149 show distributaries, not drawn to scale, in embankment and excavation.*

Article 50. Submerged Dams.

Submerged dams, also called sub-soil dams, are frequently constructed, across and under the beds of streams, with the object of intercepting the subterranean flow of water in channels whose beds, after rain ceases, soon become dry on the surface.

In the construction of a submerged dam a trench is excavated through the sand and gravel down to the impervious material underlying them. After this the trench is filled with puddle, or a wall of masonry or concrete is built up to, or nearly as far as, the surface of the bed of the channel. Then, if there is no leakage, the water rises to the surface and is conveyed away, by either an open channel or a pipe.

If the rocky sides of a channel or its bed are fissured, or if the bed-rock is porous, it is almost certain that no water can be intercepted. The foundation for a submerged dam should be, in every respect, as sound and impervious as that of a reservoir dam, but too often this has not been the case in Southern California, as the numerous failures of submerged dams there prove conclusively.

Colonel Richard J. Hinton states†:—

* Irrigation by Rajbuhas (Distributaries) by Lieutenant W. S. Morton.
† Irrigation in the United States.—Senate Report.

"It is first ascertained by sinking shafts across the channel whether water is thus passing subterraneously. This will be observable in some cases by floating substances traversing the shaft, but if the flow is very slow it may not be detected by this means, and coloring the water with a dye will show it by a replacement of the colored, by pure water passing through the shaft. A subterraneous water flow is frequently brought to the surface by impervious strata traversing its course. Localities in which this occurs are the best sites for weirs. It is not probable that such natural bars are to be found in the plains, far removed from the sources of supply, and to produce them artificially in such situations would necessitate very deep and probably very extended walls. The trial shafts should therefore be made where the valley is well defined in character.

"Of course these submerged dams can only bring water to the surface of the channel, where the latter is of sand or gravel, through which the water would rise, forming an artesian supply. Where the surface of the bed is of sand, in which the water could be again lost, the elevated water would of course be diverted to an impervious channel provided for it. Where such subterranean water can be intercepted a considerable supply might be expected for some months after the water ceased to flow previous to the interception, for doubtless in many cases a considerable proportion of the rainfall is absorbed and given off gradually to subterranean strata."

Article 51. Construction—Canal Dredger.

The following brief notes are given, chiefly for the information of engineers in other countries, outside of America, and who have never seen American methods of construction.

"To begin with the simplest kind of construction,* that of field ditching: the farmer does this, as a rule, with his plow, with which he can easily run a ditch of a few inches capacity across his field. If he intends to widen it while keeping it shallow, he employs the ditch plow, which consists of a blade suspended behind the shear so as to push the earth which it cuts to one side. In many soils this is found to be an invaluable implement. When the work is more roughly done, what is known as a V scraper is brought into play. This varies from a mere log of wood with a couple of old spade heads nailed in front forming a sharp prow, which is its rudest form, to a triangle some six feet wide at its wooden base, from which proceeds two long iron blades forming the acute angle. Its use is always the same. It is drawn by horses and steadied by the driver's weight so as to push the earth outwards from a simple plow furrow, or series of furrows, and thus form a ditch. When this is over six feet wide a "side wiper" is generally substituted, which is a long iron blade, lowered from a frame which rests upon four wheels, so that when drawn by a powerful team it slants the plowed soil to one side. In light soils and for large ditches, an elaborate machine is used, which not only plows the earth, but takes it up and shoots it out upon the banks a distance of ten or twelve feet on either side, at the rate of from 600 to 1,000 cubic yards per day. But the implement most in use for operations of any extent is the iron "scraper," well known in Victoria as the "scoop," which is found in many forms, sometimes it runs sledgewise, sometimes upon wheels, and ingeniously fitted so as to be tilted without effort. For a long pull, wheels are considered best, and for steep banks runners have the preference. The kind

*Irrigation in Western America, Egypt and Australia by the Honorable Alfred Deakin, M. P., Victoria.

of soil to be moved and worked upon, and the length of haul, are always taken into account in determining the class of scoop used. There is another implement known as the buck scraper, which for ordinary farming use in light soils, and in practiced hands, accomplishes remarkable results. It consists of a strong piece of two inch timber, from six feet to nine feet long, and one foot three inches high, with a six inch steel plate along its face projecting two inches below its lower edge, and is strengthened with cross-pieces at the back, where there is a projecting arm, upon which the driver stands. Like the ordinary scraper it is also found on wheels and runners, and in many patterns, and is drawn by a pair of horses. Instead of taking up the earth as the scraper does, it pushes the soil before it, and, when under good command, does such work as check-making, ditch excavating, or field levelling, in sandy soils, with marvellous rapidity."

A novel method of excavating a canal has been adopted in Northern California. It is illustrated in Figure 190, which is a view of a canal dredger invented and operated by the San Francisco Bridge Company. It is now in use digging the main canal of the Central Irrigation District, which is fifteen feet deep, six miles long, sixty feet wide at the bottom and one hundred feet wide at the top.

The following description of this machine is taken from the *California Irrigationist* of August 1, 1891:

"The bid of the Bridge Company for this work was about thirty thousand dollars lower than that of any of their competitors. It was the only firm of contractors who figured on doing this work by machinery; the other contractors estimated on doing it by the old method of scrapers and horses.

"The machine, a cut of which is herewith presented,

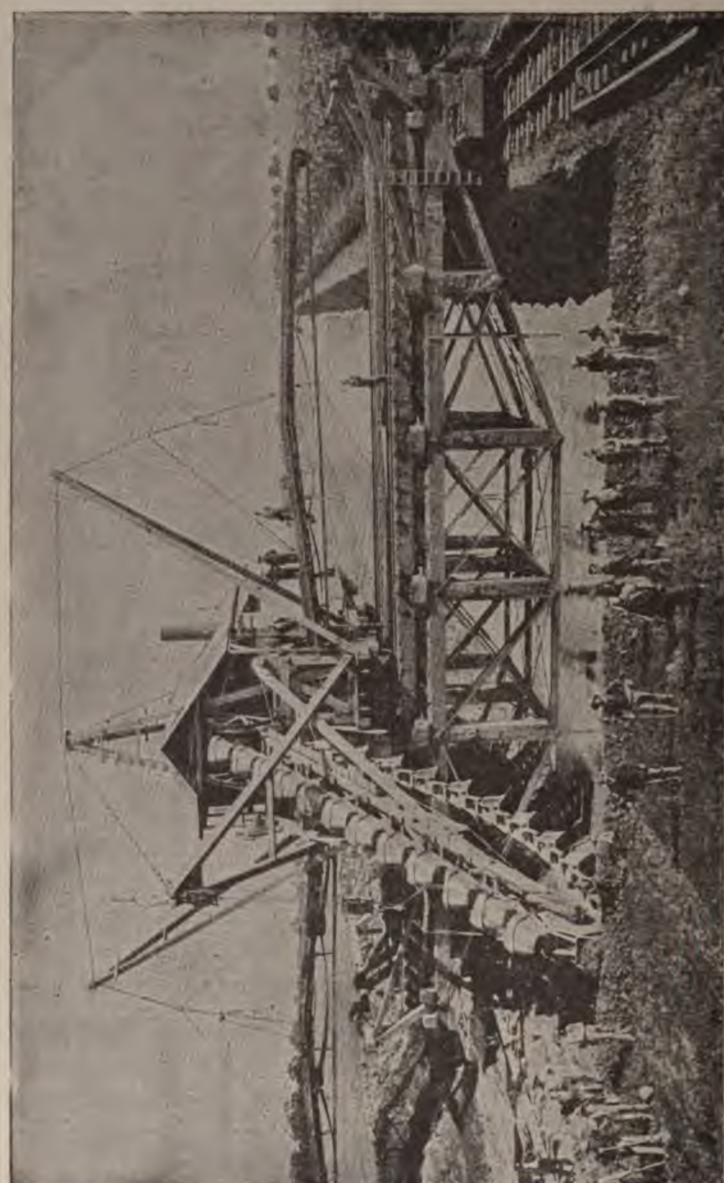


Fig. 190. Canal Dredger.

was conceived, invented and designed by the Bridge Company for the carrying out of its contract, and it has proved remarkably well adapted to the work and is in every way a success. It would have been absolutely impossible to have excavated this ditch in the old way with scrapers, owing to the presence of water, which in the summer months stood about two feet deep in the ditch, and in the winter months was often as deep as five to seven feet. The designers of the machine anticipated this condition, and ingeniously arranged the machine to rest on the original ground at the foot of the spoil bank at the top of the ditch, and not on the bottom of the ditch as steam excavators usually do. A standard-gauge railroad track is laid on either side of the ditch, as may be seen in the cut; on each of these tracks are located three very heavy railroad trucks, similar to flat cars only shorter; on these trucks are rested the three trusses that span the ditch and carry the car, which runs on double track standard-gauge, and on which is located all the excavating and transporting machinery, as shown in the illustration. The cars on the tracks on either bank are moved forward eight or ten feet at each shift by means of wire ropes worked by steam drums, fastened to "dead men," or anchors fixed in the ground 100 or 200 feet ahead of the machine; then the excavating chain and buckets are lowered, by means of another steam gypsy, until the buckets come in contact with the ground, and the car is started across the transverse track by means of another steel cable worked by a steam drum; and the buckets, as the machine passes transversely across the ditch, take a cut off the top of the ditch of the whole area of the eight or ten feet which the machine moved forward, and when the machine arrives at the other side of the ditch, the boom is again lowered and the car started back, and another cut is excavated by the buckets.

"This operation is repeated until this section of the ditch is taken out clear to the bottom, then the ladder is raised by a steam drum so that the buckets clear the ground, and the side cars are again run ahead another eight or ten feet as before, and the buckets are again lowered until they come in contact with the ground, and the car started on the transverse track again. The buckets dump or discharge into a hopper, the bottom of which is inclined and reversible, and the material after falling into a hopper falls down over this incline bottom, which delivers it on the rubber belt conveyor, which carries it to the spoil bank. When the machine passes the center of the ditch, the bottom of the hopper is tilted to the other side and the material is thrown on the other conveyor, which delivers it on the opposite bank."

Article 52. Water Power on Irrigation Canals.

Water power is utilized to a far greater extent on the canals of France, Spain and Italy than it is on the irrigation canals of India or America.

The Hon. Alfred Deakin, M. P., gives an account of the application of the water power of an irrigation canal for the purpose of irrigating land on a higher level than the canal.* He states that:—

"On the Cigliano Canal, above Saluggia, is the only instance in Italy in which the motive power of water is used on a large scale in connection with irrigation. Three canals, the Rotto carrying 565 cubic feet per second; the Cigliano, carrying 1,766 cubic feet per second; and the Ivrea, carrying 600 cubic feet per second, round the side of a steep hill, one above the other in the order named. The waters of the highest, the Ivrea, feed the

*Irrigation in Western America, Egypt and Italy.

Cigliano, while the waters of the Cigliano, by a fall of twenty-one feet into the Rotto, generate a sufficient force to lift part of the waters which have been poured from the Ivrea to the crest of the hill sixty-two feet above it, and 130 feet above the Cigliano. From this height it is distributed over the surrounding plateau, which is 164 feet above any natural water supply. The first cost of the machinery employed was $140,000, and a further outlay of $20,000 was incurred before it could raise twenty-five cubic feet per second, the volume desired. The working expenses are small, but capitalizing the rent paid to the government for the water, the total cost of the work amounts to $200,000, or nearly $8,100 per cubic foot per second. From such illustrations it is evident that, ingenious and economical as many of their works are, the Italians appraise the value of water almost as highly as the Southern Californians, and are prepared to undertake the most expensive and difficult works where it cannot be obtained without them."

To show the extent to which the water power of irrigation canals has been utilized in other countries the following examples are given:—

The Crappone Canal in France, having a capacity of from 350 to 500 cubic feet per second, moves thirty-three mills situated on its course.

On the Marseilles Canal in France, the owners of one hundred and seven mills use the fall of the water in the canal for motive power, developing about 2,000 horse-power. Probably over twenty per cent. of its revenue is derived from this source, and the tariff for the use of the water for motive power at the numerous falls along the canal was, a few years since, $40 per horse-power per annum. A horse-power was fixed at 43,296 pounds of water falling through one foot per minute. The water, after being used for motive power, had to be returned

to the company's canal at a lower level, and not appropriated for any other purpose, except by special arrangement. When the water was not used by subscribers for irrigation, it could be employed temporarily for motive power at the rate of $5 per horse-power per month.

On the Verdon Canal in France, there existed, some time since, at the numerous falls along the canal, water power to the extent of 2,000 horse-power, which was fixed to be let at $40 per horse-power per annum.

The water power of the Henares Canal in Spain, has been estimated at 3,630 horse-power for nine months, and 1,450 horse-power for the rest of the year.

Article 53. Cost of Pumping and of Water.*

Fearing the failure of the immense masonry barrage (described at page 97), which crosses both branches of the Nile, a short distance below Cario at the head of the Delta, upon which the supply of water to the perennial canals largely depends, the Government in 1885 made an agreement with the Irrigation Society of Behera, by which it undertook to pay $210,000 a year for thirty years for a supply up to a certain level, with a maximum of about 2,604 cubic feet per second at Low Nile, lifted by two powerful sets of steam pumps into the Western Canal or Rayah Behera. The weir has since been rendered secure, but the agreement indicates the value of water and the difficulty of obtaining it, even in parts of Egypt. Owing to the defective alignment of some, and the silting up of other canals, the task of raising the water a second time from the channels to the fields has been cast upon a large, if not the largest, body of the cultivators. In 1864, according to Figari Bey, the

* Irrigation in Western America, Egypt and Italy, by the Honorable Alfred Deakin, M. P. of Victoria, Australia.

number of sâkiyehs or wooden water-wheels used in Central and Lower Egypt was about 50,000, turned by 200,000 oxen and managed by 100,000 persons, who watered 4,500,000 acres. The water-wheels are of several varieties, costing on the average, with the well, $150 each, that most in use sufficing for five acres, or ten acres if worked day and night, and employing three bullocks and two men on each shift.

In the estimate of Figari Bey, some steam pumps were probably overlooked; for twenty years later there were 2,000 of these at work in lower Egypt, with coal ranging from $10 to $20 per ton. It can now be bought in Alexandria for $5 per ton. The cost of steam pumping is about $1.50, but the price at which it can be hired varies from $2 up to $5 per acre. If paid in kind the charge is often one-fifth of a cotton, and one-quarter of a rice crop, as the latter requires more water. A ten-horse power engine gives an ample supply for 100 acres during the season. There are also "shadoofs" (Egyptian water-lifters or swing buckets) innumerable in constant employ, which require six men to keep watered one acre of cotton or sugar-cane or two of barley. "If the thin deposit of mud left by the departing river is kept moist its value remains at par. If the hot sun is allowed to play upon it unopposed, it soon becomes baked, and curls up into tiny cylinders; then, breaking into fragments, it falls dead and worse than useless. Therefore, the process of irrigation must begin at once. The rude sâkiyeh and the ruder shadoof are kept going night and day, and give employment to tens of thousands of people, and cattle as well.*

The cost of this incessant labor cannot be estimated. "There is the greatest dearth of accurate statistics,"† and especially of statistics which would show what is

* "The Modern Nile," G. L. Wilson, Scribner, September, 1887.
† Public Works Report 1884.

paid for the water and what is produced by it. Though twenty-eight taxes were repealed in 1880, and others have been removed since, the taxation now ranges from \$5 to \$10 per acre, and sometimes, in Upper Egypt, amounts to more than twenty per cent. of the gross annual value of the farm. Over 1,000,000 acres of the irrigated land belongs to the State, the Fellâhin upon them being its tenants, with a life interest and a title to their improvements; half as much is included in great estates, while the balance is in the hands of small proprietors. Omdehs, or notables, and sheiks, who control the village communes, often own estates of 1,000 or even 2,000 acres, but the holdings of the great majority of their constituents, who are working proprietors, are very small. The Crown tenants, of course, pay rent, but all pay a "land tax" of from \$1 to \$8 per acre, which might be more properly named a water rent, as no tax is levied if no water is given. It is clear that, if in addition to the taxes, there is the cost of pumping, and four months' labor taken by the *corvée*, the produce must be great to yield any profit to the cultivator. The cost of the crop, including taxes and pumping, averages \$25 per acre. The value of land averages \$60 per acre in Upper Egypt, and from \$100 to \$125 in Lower Egypt, but it not unfrequently reaches \$100 in the one and \$300 to \$350 in the other. Its variation may be judged from the fact that rents run from 50 cents to \$50 per acre. Labor, of course, is plentiful and cheap—wages averaging from 32 cents to 14 cents per day—but, on the other hand, the agricultural implements employed are of the most primitive character; the plough used is made on the same model as is delineated upon monuments thousands of years old, and the Nile mud, though freely and easily worked after the subsidence of the water, requires constant attention throughout the year.

Article 54. Maintenance and Operation of Irrigation Canals.

The defective design and construction of the greater number of irrigation canals in this country, have been already referred to. But this is not all, for the maintenance is equally bad. Repairs are seldom carried out in a thorough and workmanlike manner. Weeds, bushes, and even trees, are allowed to grow in, and obstruct the channels. Brush is allowed to collect and form obstructions to the flow. In some places the channel gets silted up and bars are formed, and in other places extensive erosion takes place. A great loss of water takes place from defective banks and leaky flumes. The channel, in some cases, floods large areas of land, causing serious loss of water. The side slopes and grades of the canals are allowed to take care of themselves, and when breaches occur in the banks, the repairs are done in a hurried and slipshod way. Anything is good enough to fill in the breach in the banks.

When drops are washed out they are seldom replaced, then retrogression of levels takes place, and the surface of the water gets lower and lower, until the velocity of the current has adjusted itself to the material cut through, and the channel has established its regimen. In consequence of the scouring out of the bed of the channel, the sub-soil water passed through is lowered, causing in some cases, great injury to the land. If the channel has fall enough, and it usually has too much fall, it is assumed that the canal can take care of itself.

For the proper conservancy of the canal it should be closed once a year, at least, for repairs. Stakes should be set in the bed, to grade, and the silt removed to this level. The banks should be trimmed up, and all weeds, brush and other obstructions removed. Weirs, headworks, bridges, flumes, sluices, drops., etc., should be

put in thorough repair. This will be found the cheapest method in the end, and, by this means, the water can be kept in better control, and the canal worked to much better advantage, than when it is allowed to fall into bad repair.

Telephone service should be established along the line of the canal, and a roadway on one bank will be found useful. The official in charge, whether engineer or superintendent, should be informed every day by the patrolman of the quantity of water flowing into the canal at the head works, and also the quantity discharged at each irrigation outlet. He should also be immediately informed of any breach in the canal banks, or anything else likely to cause damage, or a partial obstruction to, or complete stoppage of irrigation in its main or distributary channels.

The Indian, Egyptian and Italian Irrigation Canals are closed, at least once a year, for clearance of silt and repairs in general. Some of the Indian canals are closed for about six weeks annually. The Naniglio Grande, or Grand Canal of the Ticino in Italy, is closed twice a year. An instance of frequent closing is given on one of the small Indian canals. In the Irrigation Revenue Report of the Bombay Presidency, for 1889-90, it is stated that:—

"The Palkhed Canal was closed six times during the year for clearance of silt, aquatic plants," etc.

Mr. Walter H. Graves, C. E., has made some remarks on this subject which will be found useful here. He states:—*

"Maintenance and superintendence are matters of considerable importance in the management and success of any enterprise, but especially important in irrigation

*Irrigation and Agricultural Engineering in Transactions of the Denver Society of Civil Engineers for June, 1886.

plants, for obvious reasons. The roadbed and rolling stock of a railroad might be allowed to deteriorate for some length of time without seriously impairing the operation of the road, but deterioration in the head-works and channel of a canal means speedy paralysis.

"The sources of impairment of canal property are:—

"*First.* As to the channel. The water itself carried by the canal, by the erosion of the banks and channel, and the filling of the channel by the deposition of sediment.

"This is a process of self destruction.

"*Second.* From the storm or flood water. The denuding of the banks by the erosive action of the elements is a constant source of destruction, although it is a comparatively small item. From the very nature of the alignment or location of the canal it must intercept to a greater or less extent the slope, and consequently the drainage of the country it traverses. If ample provision is made to transfer the flood or drainage water across the canal by means of flumes, culverts, etc., destruction from this source is largely prevented. But, as a rule, provisions of this character are wholly neglected. In many cases, where the slope of the country is sufficient, there is no upper bank to the canal, and the drainage channels are allowed to empty directly into it. Thus the surface water of the entire country above the canal is gathered into it, and the result is, in such cases, a constant rebuilding and repairing of banks.

"*Third.* The destruction of the channel, and especially the banks, by the range cattle, which can only be prevented by fencing the canal.

"The deterioration in the structures of a canal are:

"*First.* The head works. If these are of such a character as to be proof against the strain and force of the annual floods, and to meet the requirements of the

wide range of the fluctuations of the average mountain stream they must be very complete and expensive structures, and quite out of the reach of the average company. The class of work usually adopted, however, is such as to make the liability of destruction and the cost of repair important items in the subject of maintenance.

"*Second*. Applying to all structures is *decay*. Timber intervening between water and earth, and alternately soaked and dried, is particularly subject to decay, and the life of wooden structures can scarcely be prolonged beyond six or eight years.

"*Third*. Incendiarism. Strange as it may appear, this has proven, in the experience of the larger canal companies, an item of considerable importance.

"The subject of maintenance directly involves that of superintendence. An ignorant or an indifferent superintendent can increase the cost of maintenance many fold.

"Where incipient disaster may easily and cheaply be curtailed by intelligent vigilance on the part of the superintendent, serious calamities often occur by reason of his carelessness and ignorance. As a case in point, a leak of apparently insignificant proportions was allowed to exist for some time through the embankment adjoining the head-gate of one of the largest canals in Colorado, when it suddenly assumed a magnitude beyond control, until it had almost completed the destruction of the head-gate, a structure costing several thousand dollars. In this case as in many others similar, bad superintendence was credited to bad engineering.

"It seems to be quite the custom in Colorado to select canal superintendents from among any class of men except engineers, the very men best fitted by experience and training for such work."

In India the irrigation canals are always under the

control of the engineers of the Public Works Department. They control the movement and distribution of the water, and carry out all repairs and additions to the works. In order to know at all times the quantity of water available they have numerous gauges, the readings of which reach the controlling office every day, and it is a rule that he should write them into his gauge book with his own hand.

There is one arrangement, however, which, though it works well in India, is not suited for this country, that is, executive and assistant engineers engaged on the canals there, usually have powers of an assistant magistrate for the protection of canal property.

The following extract is pertinent to this subject:*

"It is too commonly supposed that when the canal is once constructed, there remains little for the executive engineer to do worthy of a man of any experience, ability or education. This is a very great mistake. There may be no great works left to construct, but there are sure to be many small ones requiring much experience and precision to execute properly. There are many points of the purest science still undetermined, such as the true formulæ for the discharge of large bodies of water in open channels, or over weirs, the amount of loss by percolation and evaporation; the effect on the velocity of a stream of a large percentage of silt carried along. The executive engineer may have besides, to train and do battle with rivers of great size, or the not less troublesome hill torrents. He may have in his charge a series of weirs which have to be constantly watched and protected, while repairs, often of the most important character have to be executed within the space of only a few days when the canal can be closed.

*Roorkee Treatise on Civil Engineering.

Alongside of his weirs he may have locks to superintend. His rajbuhas (laterals or distributaries) ought to be a source of constant interest, requiring extension and improvements, while he will find, as he goes on irrigating, that drainage has to be attended to and artificial cuts to be laid out, to correct the over-saturation which only the best administration can prevent from taking place, and to ward off the malaria which over-saturation produces.

"Besides all this, no man should consider it beneath his attention to exercise almost independent control over a large body of water, bringing in a revenue every year of $200,000 to $300,000, and also of being a source of wealth to the country of at least four times that amount.

"He should possess a general knowledge of the agriculture of the district, and know at what season the various crops most want watering, and what soils most require it. If he is fond of forestry, he will find room for gratifying his taste in cherishing and extending the plantations along the banks of his canal, and may render lasting benefits to the country by the introduction of new trees.

"Among lesser matters, he may turn his attention to utilizing the water power of his canal, a subject which must claim attention as the country progresses. If the above subjects do not possess sufficient interest for the engineer, he had better choose some other line than the irrigation department.

"Nor ought he to look for employment on a running canal if he is not prepared for a life of constant moving about, at all seasons of the year. He must expect but little of the pleasures of society, or domestic life, and be prepared for many a long, hot day, by himself, in the canal inspection house."

Article 55. Methods of Irrigation.

The methods of irrigation are generally classed under four heads, as follows:—

1st. Flooding.

2d. By distribution through furrows or ditches.

3d. Sub-surface irrigation by pipes.

4th. Sprinkling.

Of the four methods mentioned, only the first two will be referred to in the following pages, as almost all irrigation on a large scale is carried on under these heads.

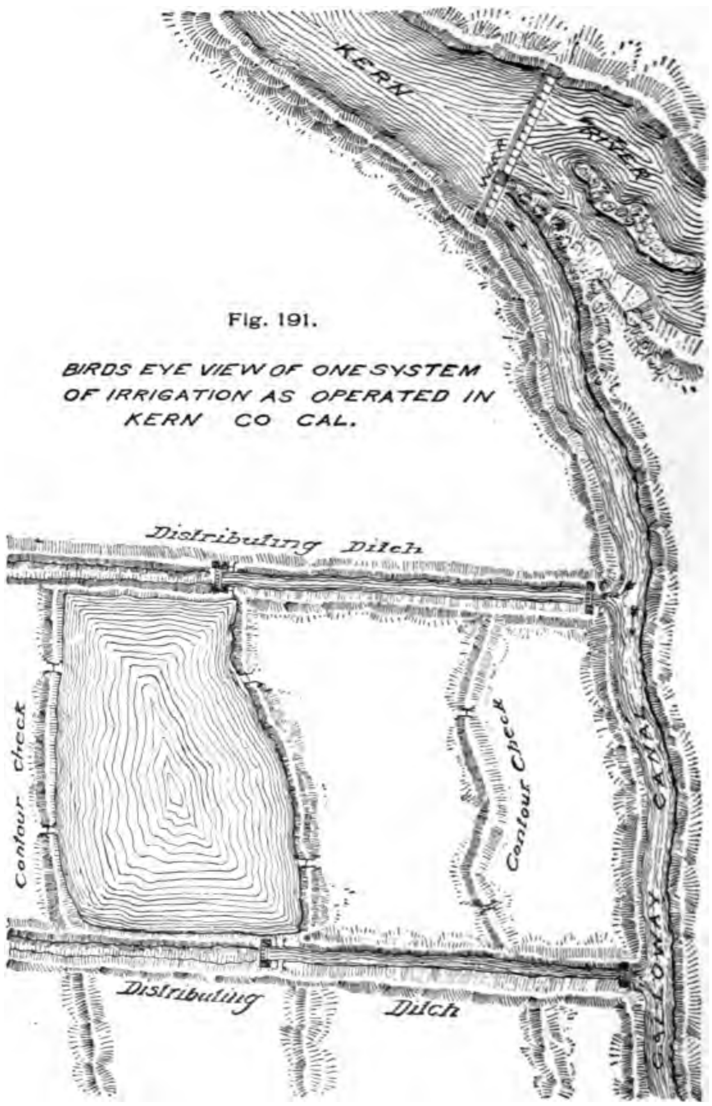
Of the above four methods, flooding is most generally practiced, and on the most extensive scale. The flooding is usually done in embanked compartments. These compartments vary in size. In India, they are sometimes as small as 400 square feet, whilst in Egypt, they are often several square miles in extent.

The following, on methods of irrigation, is compiled mainly from a paper in the Minutes of Proceedings of the Institution of Civil Engineers for 1883, by P. O'Meara, C. E., on *Irrigation in Northeastern Colorado*, and also from a paper by the Hon. Alfred Deakin, M. P., of Victoria, Australia, on *American Irrigation*.

FLOODING.

The easiest, simplest and cheapest method of irrigation is by flooding. By this method, the water is directed to cover the whole area under cultivation to a depth varying according to the crop and the quality of the soil. This plan is the most wasteful of water, but cannot be avoided in the cultivation of cereals. The only work it involves in the field is that necessary to permit an even flow of water. With a regular slope this work is sometimes trifling, but, as a rule, some preliminary outlay is required for leveling irregu-

larities, or else providing for the equal distribution of the stream from points of vantage.



To secure the highest degree of economy under the

flooding method, inequalities are removed from the surface of the land, which is then divided by small raised mounds, called "checks," into compartments, each of which is connected with a lateral or branch drain, leading from a lateral by one or more rudely constructed sluice boxes, or other cheap contrivances. The objects of these compartments are threefold, namely:—1, To check the water and to cause it to flow laterally; 2, To arrest the flooding as soon as the amount supplied is sufficient for moistening the soil to the extent deemed beneficial; 3, To diminish the inequality in the depths moistened, which necessarily arises in the circulation of water from a central point.

Figure 191 exhibits the distributing ditch taken from the main canal, the gates leading from the distributing ditch to the compartment.* The compartment flooded is the third from the main canal, and in case the two upper compartments were first flooded, their surplus water would flow through the gates shown in the checks, into the third compartment. Small gates are shown in the three checks for draining the compartments when it is deemed they have had sufficient water.

The smaller the compartments the less will be the resulting inequality, but the greater the expense of constructing and the labor of using them. Lands nearly level and lands with retentive soil admit of the largest compartments, with a given margin for inequality of moistening. The maximum of size is perhaps obtainable when the slope from the point of application is about 1 inch in 100 feet. On nearly level lands the size of the compartments may be directly proportioned to the volume of water in application. The extent of this volume is limited by the difficulty of controlling it,

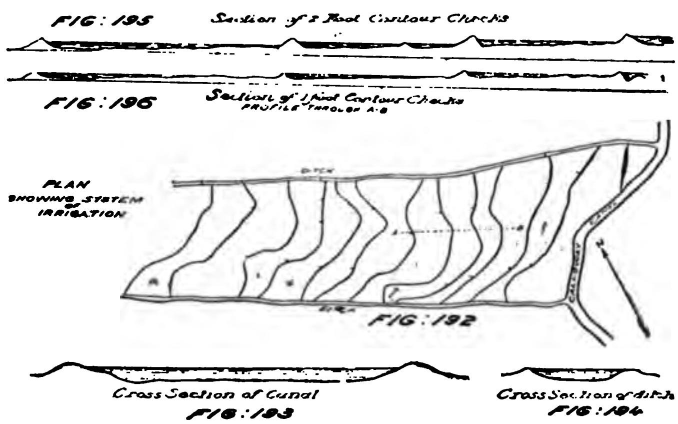
*Report of the Senate Committee on the Irrigation and Reclamation of Arid Lands.

and the damage it would do to the soil or crop if too large. Laterals of three or four cubic feet per second for broken land, and of six or seven cubic feet when the land is unbroken, are manageable under favorable circumstances, by one irrigator, although those which are in use where compartments have been tried in Colorado are much smaller. The whole of the volume in application may be admitted into one compartment through several openings, or into several compartments through one or more openings. In the former case the compartments may be larger, because the inequality of absorption depends on the time of flooding. This, to come within the margin fixed for inequality of absorption, must, in the absence of statistics for different soils, be arrived at by a tentative process, and the size of the compartments then proportioned to the volume or volumes in application.

When the fall is slight, shallow ditches are run, in Colorado, from 50 feet to 100 feet apart in the direction of the fall; when the land is steeper they are carried diagonally to the slope, or are made to wind around it, and from there, by throwing up little dams from point to point, the whole field is inexpensively flooded. When the fall is still greater and the surface irregular, ridges are thrown up along the contour lines of the land, marking it off into plots called "checks," on the whole of the interior of which water will readily and rapidly reach an equal depth on the contour line. When one plot is covered the check is broken and the water admitted so as in the same way to cover the next plot.

Figure 192 shows the contour checks beginning at the main canal, and compartments supplied by a ditch or distributary running almost parallel on each side of the compartments. Figure 193 shows a cross-section of main canal; Figure 194, a cross-section of distributary,

and Figures 195 and 196, cross-sections showing checks.* The ridges, checks or levees must have rounded crests and easy slopes, or else they interfere with the use of farming machinery, such as plows, headers, etc. By



means of diagonal furrows and checks, remarkable results are obtained, even in very broken country. By their means it is claimed that, in Colorado one man can irrigate twenty-five acres per day. Where checks have not been used upon ground with an acute incline the water has soon worn deep channels through it, utterly ruining it for agricultural purposes; or again, where the water has been allowed to flow too freely, the consequence has been that all the fertilizing elements of the soil have been washed away. In flooding, the aim is, therefore, to put no more water upon the land than it will, at once and equally, absorb or can part with without creating a current sufficient to carry off sediment. The neglect of these precautions has caused the abandonment of several settlements made in Utah before the art of Irrigation was properly understood.

*Report of the Senate Committee on the Irrigation and Reclamation of Arid Lands.

Both the depth and number of floodings are varied according to soil and crop. With a clay the waterings are light and frequent, while with a sandier quality they are heavier and rarer. Much, too, depends upon the distance and nature of the sub-soil. There is considerable uncertainty with regard to the measurements given for flooding. It is sometimes so low that it will give a depth of only two or three inches, and at other times it will give a depth of five to ten inches at a single watering. There are cases in which as many feet have been used. The number of waterings is best determined by the crop itself, and the most skillful irrigators are those who study its needs and take care to supply these needs, without giving an excess of water. The quantity used alters, therefore, from season to season, so that only an average can be given. See Article 58.

In Colorado, where water is used more lavishly than in any other State, some good judges have agreed that an average of 14 inches should be ample, and this is certainly not too low. Where the soil is liable to become hard, and will retain moisture, wheat is often grown with two floodings, one before the ground is ploughed and the other when it is approaching the ear. When two waterings are given after sowing, one is given when the wheat commences to "tiller," and the other when it reaches the milky stage. Where irrigation does not precede the plowing, it is postponed as long after the appearance of the crop as possible. Sometimes wheat has three, or even as many as four, floodings, but this is unusual, as over-watering occasions "rust." Experience shows that it is easy to exceed the quantity required by the crop, and that every excess is injurious. Extravagance is the common fault, so much so that the most successful irrigators are invariably those who use the least water. The less water, indeed,

with which grain can be brought to maturity, the finer the yield.

Colonel Charles L. Stevenson states, with reference to the methods of irrigation in use in Utah:—*

"Each farmer has canals leading from the main one to every field, and generally along the whole length of the upper side of each field. Each field has little furrows, a foot or more apart and parallel with each other, running either lengthwise or crosswise or diagonally across, as the slope of the land requires. Into these furrows the water is turned, one or more at a time, as the quantity of water permits, until it has flowed nearly to the other end, when it is turned into the next furrows, and so on until all are watered.

"This is the usual custom, but where the soil is made of clay this method is not so good and another is used. This method is to throw up little embankments six inches high around separate plats of land that are of uniform level, and turn the water in until the plat is full to the top, when the water is drawn off to the next lower plat, and so on to the end. This enables the water to soak in more and so does more good, but where the soil is porous, as is generally the case, it is not so good a method as it wastes water."

FLOODING IN INDIA.

In India, and also Egypt, flooding is universally practiced. There are two methods adopted in India in supplying water for irrigation, known as *flush* and *lift*. In flush irrigation the water flows by gravitation on to the land to be irrigated. In lift irrigation the water reaches the land at such a low level that it cannot flow over the surface of the land to be irrigated. This

* Irrigation Statistics of the Territory of Utah, by Colonel Charles L. Stevenson, C. E.

necessitates power of some kind, usually manual labor, to raise the water sufficiently to enable it to flow over the land. It is, therefore, to the interest of the irrigator to economize water, and in view of this fact the officials of the Ganges and Jumna Canals charge for lift irrigation only two-thirds of the rates charged for flow.

The proportion of flow to lift irrigation, on the Sone Canals, in Bengal, in 1889–90, was 96.3 to 3.7. During the same period on the Mazzafargarh Canals, in the Punjab, the proportion of flow to lift was as 96.1 to 3.9, but on the Shahpur Inundation Canals, in the same Province, the proportion of flow to lift irrigation was 85 to 15. There is usually more lift irrigation on inundation canals than on perennial canals.

So great was the loss from waste of water in India, some years since, that it was seriously proposed to supply all the water at such a level that it should be raised some height, however small, in order to bring it to the surface of the land to be irrigated. It would then be to the interest of the irrigators to prevent waste, and the duty of water would, in this way, be materially increased.

FURROWS.

Peas and potatoes are not irrigated by flooding, but from furrows four feet to ten feet apart, and this is found the most economical and most successful system for vines and fruit trees. The direction of the furrows is chosen so as to give a fall of from one inch to three inches per 100 feet. The expenditure of water is much less under this than under the flooding method. When the furrows are deep and narrow the practice is similar in principle, though less effective than the pipe method of irrigation, which will be described further on. Irrigation can, in fact, be carried on without flooding the intervening soil, moistening in the latter case taking place beneath the surface, and losses from evaporation

being thereby largely diminished. It is evident that the depth of the furrows should be in some degree proportioned to the depth of the roots of the crop cultivated. Figure 197 illustrates how land is irrigated by furrows.*

Fig. 197. Plan Showing Method of Furrow Irrigation.

Under the flooding system the ground, if not protected from the sun, cakes quickly. When the water is run down furrows drawn by a plow between the plants, this caking is avoided and the water soaks quickly to the roots. When flooding was practiced in orchards it

*Irrigation by W. H. Graves, C. E. Transactions of Denver Society of Civil Engineers, 1886.

was found to bring the roots to the surface and enfeeble the trees, so that they needed frequent waterings.

Sometimes the furrows feed a small hole at the foot of the tree, from which the water soaks slowly in. When this is done mulching is found desirable over the hole to reduce the loss by evaporation. The general rule is to protect the trees by small ridges, so that the water does not affect the surface within three or four feet of them. The simple furrow, however, is most generally in use.

Oranges are watered three or four times in summer; vines once, twice, or often not at all after the first year or two; and other fruits according to the caprice of the owner, the necessities of the season and the nature of the soil, one to four times. It is impossible to be more exact.

An even greater difference, comparatively, in the quantity of water used obtains in the furrow irrigation of fruit trees and vines, than has been noted in regard to cereals. To such an extent does this prevail that, not only do districts differ, but of two neighbors who cultivate the same fruits in contiguous orchards, having exactly the same slope and soil, one will use twice or thrice as much water as the other. To attain the best results the trees must be carefully watched, and supplied with only just enough water to keep them in a vigorously healthy condition.

Another all important principle, as to which there is no question, and which is testified to on every hand is, that the more thoroughly the soil is cultivated, the less water it demands, a truth based partly, no doubt, upon the fact that the evaporation from hard, unbroken soil is more rapid than from tilled ground, which retains the more thoroughly distributed moisture for a longer period.

Major Corbett published some articles in the *Professional Papers on Indian Engineering* to prove that, by the adoption of superior cultivation, the necessity of irrigation would be very much diminished in India. The native plow enters the ground for only a few inches, and below that depth there is a hard crust that prevents the water from filtering down. He contended that by breaking up this hard crust by deep plowing, and by carrying the cultivation deeper, that there would not be the same necessity for irrigation as was required after shallow plowing, for the reason that evaporation from the land would not take place to the same extent.

For the irrigation of cereals, works are required on a larger scale, proportionately, than for fruit, because in the first case the water is demanded in greater quantities, at particular times, while in the latter the supply can be more evenly distributed throughout the year, though, of course, the irrigating season with both is much the same.

Winter and autumn irrigations are growing in favor. Land which receives its soaking then, needs less in summer, and is found in better condition for plowing. It is argued that moisture is more naturally absorbed in that season and with greater benefit. Everywhere the verdict of the experienced is, that too much water is being used, and the outcry against over-saturation in summer is but one of its forms.

Article 56. Duty of Water for Irrigation.

The duty of water is that quantity required to irrigate a certain area of land. In English-speaking countries, it is usually expressed by stating the number of acres that a continuous flow of one cubic foot per second will irrigate. Thus, if a stream discharging 40 cubic feet of

water per second is all expended in irrigating 8,000 acres of land, then its duty is equivalent to 200 acres, that is, each cubic foot per second irrigates 200 acres. The duty varies from 35 to 2,200 acres per cubic foot per second.

The duty is sometimes expressed by the average depth of water over the whole land, and again, by the cubic contents, as, for instance, the number of cubic yards per acre.

The duty of water is influenced by different circumstances and varies according to the following conditions:—

1. With the character and conditions of the soil and sub-soil.
2. Configuration of the land.
3. The depth of water-line below surface of ground.
4. Rainfall.
5. Evaporation and temperature.
6. The method of application employed.
7. Length of time the land has been irrigated.
8. Kind of crop.
9. The quantity of fertilizing matter in the water.
10. The experience of the irrigators.
11. The method of payment for the water, whether by the rate per acre irrigated or by payment for the actual quantity of water used.

Payment according to the actual quantity of water used is a good method to make the irrigators use the water with economy.

Mr. J. S. Beresford, C. E., in a paper on the Duty of Water,* enters very fully into all the causes of waste of water. He states, under the heading:—

"*Efficiency of a Canal.*—Take the Ganges Canal. We

*Professional Papers on Indian Engineering, Vol. V, Second Series.

may look on it as a great machine composed of many parts, and go about calculating its efficiency in the same way as that of a steam engine. This irrigating machine is made up of four important parts, which are quite separate, and, as things stand at present, at least two of them depend on different interests. They are as follows:—

"1. Main Canal.

"2. Distributaries.

"3. Village water-courses.

"4. Cultivators who apply the water to the fields.

"Each cubic foot of water entering the head of a canal is expended as below:—

"1st. In waste by absorption and evaporation in passing from canal head to distributary head.

"2d. In waste from same cause in passing from distributary head to village outlet.

"3d. In waste from same cause in passing along village water-course to the fields to be watered.

"4th. In waste by cultivators, through carelessness in not distributing the water evenly over the fields, causing evaporation, and the ground to get saturated to an unnecessary depth in places. (See page 250.)

"5th. In useful irrigation of land.

"Our object is plainly to increase the fifth by the reduction of all the rest."

All over the irrigating districts of America, where irrigation is carried on from earthen channels, the duty is low. We see in Table 18 that the average duty in India is over 200 acres, and it is doubtful if the average in America is half of that quantity. There is one fact that may account for this great difference. In America the greater part of the land irrigated is virgin soil, and this may account for the great quantity of water used. In India, on the contrary, the land has been irrigated

for centuries and the average rainfall is greater than in the arid region of America. A great portion of the land now irrigated by canal water in India was irrigated from wells before the construction of the canals. Whatever the cause may be, the fact is apparent, that the duty of water in America is far below that of India.

We have already seen, in page 249, how Mr. A. E. Forrest, C. E., by simply improving the distributaries, raised the duty of water in one of the divisions of the Ganges Canal to 400 acres.

There is no good reason why such a duty of water should not be reached in many districts of America. As the area of irrigated land increases so will the value of water increase, and irrigators will then be compelled to keep their main and distributing channels in good order, to use the water at night to prevent all waste and to put no more on the land than is sufficient to mature a crop.

The following table, showing the duty of water in different countries, is compiled from various sources and includes a table given by Mr. A. D. Foote, C. E., in his Report on Irrigating Desert Lands in Idaho:—

TABLE 18. Giving the duty of water in different countries.

LOCALITY.	COUNTRY.	Duty of water.	REMARKS.
		Acres.	
Eastern Jumna Canal	India	306	E. B. Dorsey, C. E.
Western Jumna Canal	India	240	F. C. Danvers.
Ganges Canal	India	232	E. B. Dorsey, C. E.
Canals of Upper India	India	267	E. B. Dorsey, C. E.
Canals of India—average	India	250	Lieut. Scott Moncrieff, R. E.
Bari Doab Canal	India	155	F. C. Danvers.
Madras Canals (Rice)	India	66	George Gordon.
Tanjore (Rice)	India	40	Roorkee Treatise Civil Engineering.
Swát River Canal, 1888-89.	India	216	Revenue Report of the Irrigation Dep't,
Swát River Canal, 1889 90.	India	177	Punjab, 1889-90.
Western Jumna Canal, 1888 89.	India	143	" "
Western Jumna Canal, 1889 90.	India	179	" "
Bári Doab Canal, 1888-89.	India	201	" "
Bári Doab Canal, 1889-90	India	227	" "
Sirhind Canal, 1888-89.	India	180	" "
Sirhind Canal, 1889-90	India	180	" "
Chenab Canal, 1888-89.	India	154	" "
Chenab Canal, 1889 90	India	154	" "
Nira Canal, 1888-89	India	186	Bombay Report, 1889-90.
Genil Canal	Spain	240	E. B. Dorsey, C. E.
Elche	Spain	1072	George Higgin, C. E.
Lorca	Spain	2200	George Higgin, C. E.
Jucar (Rice)	Spain	35	George Higgin, C. E.
Henares Canal	Spain	157	George Higgin, C. E.
Canals of Valencia	Spain	242	E. B. Dorsey, C. E.
Forez Canal	France	140	Transactions I. C. E., vol. 65.
Canals south of France	France	70	George Wilson, C. E.
Sefi, or Lower Nile Canals	Egypt	350	London Times, 18 Sept., 1877.
Sefi, or Lower Nile Canals	Egypt	274	Russian Pasha, 1883.
Canals, Northern Peru	Peru	160	E. B. Dorsey, C. E. No rainfall.
Canals, Northern Chili	Chili	190	E. B. Dorsey, C. E. No rainfall.
Canals, Lombardy	Italy	90	Baird Smith, R. E. Including Rice.
Canals, Piedmont	Italy	60	Baird Smith, R. E. Including Rice.
Marcite	Italy	1 to 18	Columbani and Brioschi.
Sefi Canals, Southern France	France	60	Lieut. Scott Moncrieff, R. E.
Sefi Canals, Victoria	Australia	200	The Honorable Alfred Deakin, M. P.
Sweetwater, San Diego	California	500	William Fox, M. Inst., C. E. } Pipe system.
Pomona, San Bernardino	California	500	William Fox, M. Inst., C. E. } Pipe system.
Ontario	California	500	William Fox, M. Inst., C. E. } Pipe system.
California	California	80 to 150	Earthen channels.
San Diego	California	1500	James D. Schuyler, C. E.
Canals of Utah Territory	Utah	100	C. L. Stevenson, C. E.
Canals of Colorado	Colorado	100	Nettleton, State Engineer, Colorado.
Canals of Cache la Poudre	Colorado	193	Prof Mead, C. E.
Canals of Colorado	Colorado	55	P. O'Meara, C. E.

Article 57. Pipe Irrigation.

Four things are necessary in order to get the greatest possible duty of water. They are:—

1. That the water should be sold or supplied by measurement.

2. That it should be conveyed to the actual point of use in impervious channels, and best of all in pipes.

3. That its use should be continuous, that is, at night as well as by day.

4. That it should be used intelligently and with a due regard to economy.

The use of pipes refers only to small supplies of water. For large supplies earthen channels are the most economical, not of water, but of money.

If the above four conditions are observed the duty of water, especially for fruit land, will be increased to a great extent, with a corresponding increase in the area of land irrigated.

The use of pipes made of plate iron, vitrified clay, concrete, wood bound with iron bands, open channels made of asphalt or concrete, and reservoirs lined with asphalt or concrete, is steadily increasing in Southern California.

The pipe system has been adopted with great success in Bear Valley, Pomona, Ontario, Riverside, San Bernardino, Los Angeles, and many other localities in Southern California, and this is conclusive proof, that the great expense attending their construction, is more than counterbalanced by the great saving of water effected by their use. By the pipe system the distribution of water is better under control, and easier managed, than by open channels.

Fred. Eaton, M. Am. Soc. C. E., of Los Angeles, has supplied the following relative to irrigation by pipes:—*

"The duty of our streams would be extended by extending the present ditches by pipe systems. Experience has taught us that by economizing the water it is not only the water that we save in seepage alone, but the distribution. The convenience that these pipe systems offer in the distribution of water is a great economizer. We find that we can get along with a half or a third the

*Quoted in Irrigation in the United States by Richard J. Hinton.—U. S. Department of Agriculture.

water that we get in running it around in ditches. It was thought that the San Gabriel was being used up by irrigating 2,000 acres, but it has been used since for irrigating 12,000 acres, and it can be increased by the pipe system. The duty of one-fiftieth of a cubic foot per second throughout the valley, under the pipe system, would be one inch to ten acres; that is, for vegetables and all kinds of crops. It depends altogether on the character of the soil. A soil that is well sub-drained, that is, composed of gravel, will require much less water. Such sub-soil is a natural drain, and for that reason water will go a great deal farther on that kind of land than it will on an impervious sub-soil. Taking the average in the San Gabriel Valley, with ten acres, you can irrigate all kinds of crops, orange trees, and all kinds of vegetables.

"The cost runs from $15 to $50 per acre. The cement pipes are not cheaper than the pressure pipes, because it requires a good many more of them, and they are not so convenient as the pressure pipes. We generally use sixteen iron. It is practically the sixteenth of an inch thick. A four-inch pipe is more difficult to make than a sixteen-inch. We put asphaltum on, but it is impossible to keep it from being knocked off in spots, and these spots rust there. We cannot inspect them closely enough to get at them all and paint them over. In ordinary soil where there is no alkali, it will wear fifteen or sixteen years. I put in pipes fifteen years ago that are doing service now. The Pasadena pipes were eleven inches with eighteen iron. That system was put in in 1873 and served up to this year. We have not many storage facilities up in the mountains, they are confined practically to the foothills and the valleys. We have to bring our water down and make our reservoir in the valley."

The following description of the pipe system of Ontario, California, is by F. E. Trask, Chief Engineer of the Ontario Land Improvement Company:—

PIPE IRRIGATION SYSTEM, ONTARIO, CALIFORNIA.

A portion of the Ontario tract of 11,000 acres is under cultivation receiving its water supply from San Antonio Cañon by means of a pipe system of main, sub-mains and laterals. The accompanying *plat* shows only a portion of the north end of the tract, the letters A A_2 A_3, B B B_3, and C C C, marking the location of mains, sub-mains and laterals respectively. Lots are 696 feet by 627 feet, or about ten acres each in area.

The general slope of the land is southeast, and the grade varies from thirteen (13%) per cent. at the north end—shown on plat—to one (1%) per cent. at the south end of the tract, which is not included in the plat.

The principal main, A A A_3, brings the water from the cañon, around the foothills, and down the same, to the head of the colony land, where, running east and west, this main supplies the laterals C C C.

Sub-mains B B B_3, or as commonly designated, the *supplementary* mains, take water from the main line, A A A_3, near the foothills and run diagonally through the colony, furnishing water to the laterals, C C C, at points some miles south of their heads, to compensate for that already expended by the latter. For example—the lateral A_3 B_3 has supplied four lots by the time it reaches B_3. At B_3 a new supply is received into the lateral A_3 B_3 from the sub-main B B B_3, which is used to irrigate land lying south of B_3.

Laterals C C C are designed to carry water without pressure and deliver the same at the highest corner of each lot. They are parallel to each other and average about six miles in length. Each line is designed to irri-

gate one tier of lots and is located three feet within the boundary of such tier, as shown on diagram. The

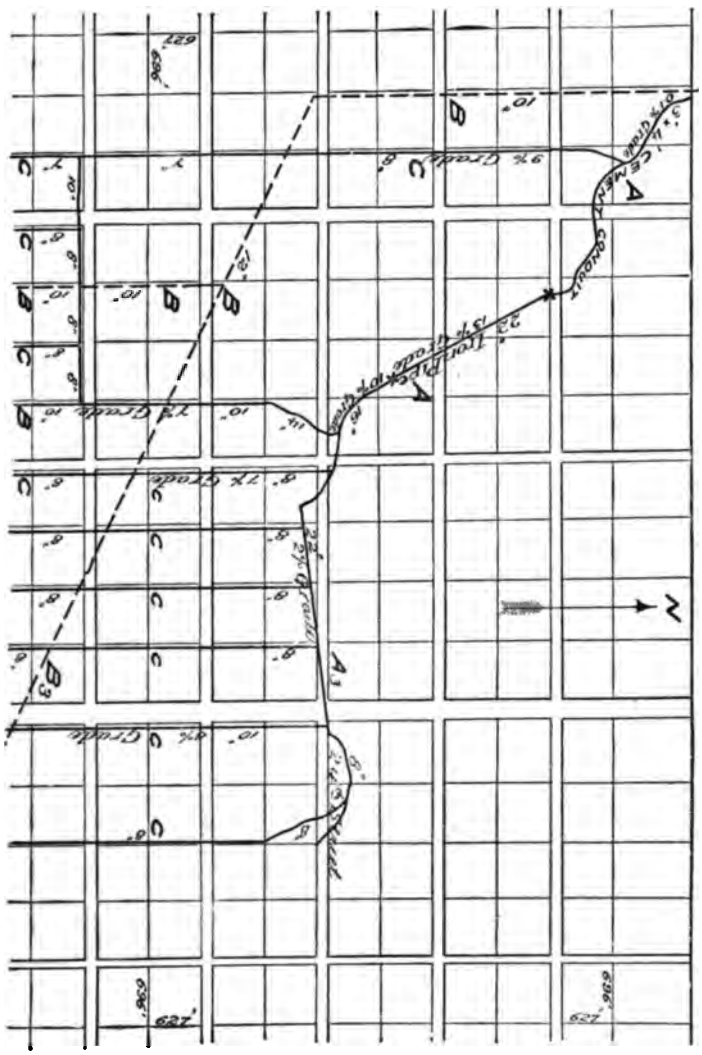


Fig. 198. Plan of Pipe Irrigation System, Ontario, California.

diameter and grades of the laterals C C C are given on the plat for the section it represents; below which the

grade constantly decreases to the south end of the tract where the grade is flattened, *i. e.*, about one per cent.; but the diameter of pipes remain the same.

Stand pipes of fourteen (14″) or sixteen (16″) inches diameter are placed in the pipe lines, *C C C*, at points where water is to be delivered to the land. In each stand pipe an iron slide gate is set; this can be dropped to close the whole pipe line or to a sufficient depth to intercept the required volume of water, as the case may be.

The greater portion of the pipe used in this system has been manufactured from cement concrete at convenient points in the tract.

Properly located and designed, the *pipe system* for the irrigation of fruit lands is much more economic than any of the older methods, and irrigators can ill afford to adopt *flume* or *ditch* systems where the topography admits of the pipe system being used.

Article 58. Number and Depth of Waterings.

The number and depth of waterings given to land vary very much in different countries.

The greatest quantity is used in the *Marcite* cultivation of Italy and the south of France, where water is poured over the meadow lands during the winter, in quantity sufficient to cover them to a depth of more than 300 feet, and where the duty has been as low, in some cases, as one acre to one cubic foot of water per second. (See Table 18.)

The other extreme is reached of a small expenditure of water by the pipe system of orchard agriculture in Southern California, where a cubic foot of water per second was estimated to irrigate from 500 to 1,500 acres.

The Henares Canal, in Spain, gives twelve waterings. Each watering is equal to about 916 cubic yards, which

gives a depth of 0.57 foot for one watering, equal to 6.8 feet in depth for twelve waterings.

The Esla Canal, also in Spain, gives twelve waterings. Each watering is equal to about 850 cubic yards, which gives a total depth on the land of about 6.3 feet.

In Valencia, in Spain, where it is very hot, wheat is watered four or five times, giving about 200 acres per cubic foot per second. In other parts of Spain a depth of two and one-half to three inches was considered ample for an irrigation, and two irrigations in the seasons were held to be sufficient.

In some of the gardens of Valencia, Spain, only from 13 to 20 acres per foot are irrigated. Here, however, there are at least two crops a year and a part is devoted to rice.

In the new canal from the Rhone, in France, the summer waterings will generally be twenty in number, given once a week, and representing a total depth of one metre or 3.28 feet.

In the south of France the time for irrigation commences on the 1st of April, and terminates on the 30th of September. The standard quantity of water adopted in the country is one litre (.0353 cubic feet) of water supposed to flow continuously for six months, per hectare (2.471 acres). This quantity of water would cover the ground to a depth of about 62¼ inches; consequently it gives fourteen irrigations, each of about four and one-half inches; twenty irrigations of about three inches, or forty-three irrigations of about one and one-half inch depth of water.

There is no fixed rule in the south of France as to the number of irrigations for such crops which require periodical irrigating, during the whole season, as this must necessarily depend, to a great extent, upon the nature of the land, whether light or heavy, whether flat

or sloping. In most cases the water is given by the companies once a week, which would be equal to twenty-six irrigations during the season. The Marseilles Canal gives the water forty-three times during the season.

Experiments, near the Bári Doab Canal, in India, showed that an average depth of 0.24 feet on the whole surface represents a thorough watering of the average soil of the district, sandy loam, and that for sandy soils 0.31 feet in depth, and therefore the amount of water necessary for an average watering of one acre is $0.24 \times 43{,}560 = 10{,}454$ cubic feet.

Wheat in a dry season requires five waterings; the first for preparing the land for plowing at 10,500 cubic feet, and four for the standing crop of 8,000 cubic feet, gives 42,500 cubic feet in all necessary for each crop of wheat that is an average depth of less than one foot.

In Madras 6,000 cubic yards of water are usually given to irrigate an acre of rice. This is equivalent to a depth of 3.7 feet.

In Colorado the expenditure of water for a single irrigation is generally reckoned at about twelve inches in depth. Of irrigations the number applied to the land in one season is about three, in exceptionally dry ones, four. The English company in Colorado has a water right equivalent to a depth of 42.84 inches.

Professor George Davidson, of San Francisco, says that the best authorities assume a depth of from 10 to 12 inches of water to the production of a crop of wheat, barley and maize, when applied in waterings of four times two and a half inches or three times four inches. The smaller of these results is almost identical with the amount deduced from observation in the great valley of California, where a rainfall of 10½ inches, fairly distributed, has insured a large crop of wheat, etc.

Colonel Charles L. Stevenson, C. E., states, with reference to Utah*:—

"Each farm generally has the right to use the water so many hours once a week or once in 10, 12 or 14 days, as the particular valley and the time of year require. The crops are supposed to get a good soaking at every watering."

General Scott Moncrieff states, with reference to irrigation in India:—

"For the wheat crop which is grown in the cold season, four waterings are quite enough, and almost no other crop requires more, except rice and sugar-cane, which are sometimes irrigated as often as twelve times, and are watered by a rainy season as well. From actual experiments in the Northwest Provinces of India, in the months of December and February, when it is by no means very warm weather, I found that one cubic foot of water per second would irrigate in twenty-four hours 4.57 acres of rough, uncleaned ground previous to plowing, and that this same discharge was enough for 5.64 acres of a well-cleaned and level field of young wheat. These results give depths of water of 5.1 inches and 4.1 inches. A safe mean in Northern India is to reckon five acres in twenty-four hours as the area to be watered by one cubic foot per second, where, as is general, the soil is light.

"We may further take fifty days as about the greatest interval there allowed to elapse between two waterings, and so we shall obtain $5 \times 50 = 250$ acres as the duty to be got out of each cubic foot per second, that is, .28 litre (.009886 cubic feet) per hectare (2.47 acres), supposing it can be used at this rate all the year round, and this is not more than has been done more than once on

* Irrigation Statistics of the Territory of Utah.

the Eastern Jumna Canal. The discharge then is measured at the head of the canal, and the water probably runs on an average more than 300 miles before it actually reaches the field to be watered. It is usual to deduct twenty per cent. for the loss by filtration, evaporation, etc., *en route*, and yet a duty as high as this has been proved attainable without making allowance for the deduction. Of these 250 acres, about eighteen per cent. usually consists of rice, and as much more of sugar-cane, each requiring a large amount of water; fifty per cent. of wheat and barley, and the rest of inferior crops, only watered once or twice. The rain, of which for the greater part falls in June, July and August, consists of about 40 inches a year—more certainly than in Castile. The heat and consequent evaporation must be considerably greater."

Article 59. Horary Rotation.

Water is supplied for purposes of irrigation:—

1. By fixed outlet or by measurement.
2. By the area of land irrigated to certain crops.
3. By Horary Rotation.

The latter method of supply will be now considered.

In order to obtain the greatest duty from water, it should be used at night as well as during the day.

An irrigating channel passes through the lands of several proprietors. A period of rotation is fixed for this channel. This period varies according to the nature of the crop, rice for example requiring a more rapid rotation than wheat. Each landowner can then have the full volume of the channel turned on to his land, once in the period of rotation, for a certain number of hours, according to the quantity to which he is entitled.

This method is applicable only to laterals or distribu-

taries, having a small discharge, which a landowner can handle with economy.

It is clear that the quantity of water to which any single employer of a canal, common to several, and regulated by horary rotation, is entitled, is in direct proportion to the total volume of the canal, and the number of hours during which he is entitled to possess it, and in inverse proportion to the number of days over which rotation extends. Hence we have the following general formula:—

$$Q = \frac{Q_1 T}{N} \text{ and } \therefore\ T = \frac{NQ}{Q_1}$$

where:—

Q = the quantity appertaining to a single consumer in continued discharge.

T = the number of hours or days during which he has the right to the whole volume of the canal.

Q_1 = the volume of channel in cubic feet per second, or any other fixed measure.

N = the number of days over which the rotation extends.

Example.—Let 10 days be the period of rotation, and the channel has a supply of 20 cubic feet per second, of which a consumer is entitled to a continuous supply of one-twentieth part or one cubic foot per second. He wishes to change this continuous for an intermittent supply:—

$$\therefore\ T = \frac{NQ}{Q_1} = \frac{10 \times 1}{20} = .5$$

Therefore, he is entitled to the full supply for half a day or twelve hours. His name is placed on the list, say sixth, and he gets the full supply turned on at a fixed hour and turned off at a fixed hour also. Arrangements can be made to have another consumer's gate opened as this one is being closed, and, in this manner, the full

supply of the channel is delivered on the land continuously.

Mr. R. E. Forrest, C. E.,* states:— "That by a good system of rotation it might be possible to remedy the loss of duty from the water not being used at night; the water could be run on at night to the more distant points. By a system of rotation the evils of supersaturation could be lessened. The water was made to run through a tract only when it was wanted and for so long as it was wanted. In some of the Ganges Canal channels the water ran only for a single day each fortnight. The water should be completely withdrawn from every tract in which it was not in active and immediate demand."

Article 60. Forestry and Irrigation.

The preservation of the forests, and the extensive planting of trees, should proceed simultaneously with the development of irrigation in this country. A stop should be put, and at an early date, to the ruthless destruction of the forests of this country, especially at the head-waters of the rivers, for if this is not done, what has happened in other countries is sure to happen here, and districts which are now fertile will, in the lapse of time, become barren wastes.

A large forest, is in fact, an immense reservoir, which slowly but surely gives out its supply for the wants of man. The greater part of its loss by percolation is again utilized to supply the streams, and it requires no dams or other expensive works. The Government send out engineering parties to locate the sites of reservoirs, whilst, at the same time, they permit nature's own reser-

* Transactions of the Institution of Civil Engineers, Volume LXXIII—1883.

voirs, the forests, to be destroyed in the interest of a few individuals.

Parts of Persia that are now desert were, within historic times, fertile lands, which supported dense populations and yielded large revenues. During long periods of time, different large armies, with their countless hosts of camp followers, passed across the country and cut down the trees for fuel. The inhabitants of the country did the same thing, and no trees were planted to replace those that were destroyed. During the existence of the forests the rain fell at regular intervals, and in moderate quantities, and, in this way, was an aid in the cultivation of the land and in maturing the crop. After the destruction of the forests the rain fell, in dense showers, at irregular intervals, thus doing more harm than good, and as a result of this the population gradually diminished until the land became a desert.

In America, an army of wood-cutters is *constantly* employed in destroying the forests. It takes a short time to destroy a forest, but many a year, equal to several generations of men, to reproduce it.

Mr. Allan Wilson, Mem. Inst. C. E., states * with reference to Southern India:—

"In former times when the tanks (reservoirs) were in good repair, trees were largely planted, and, as is always the case, vegetation attracted the moisture, and the monsoon could always be depended upon. Now, since these works have fallen into decay, the vegetation has disappeared, and the monsoon has been precarious and insufficient."

Every year we read in the press of destructive floods taking place in the old country, and these floods are almost all due to the destruction of the forests.

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* On Irrigation in India in Transactions of the Institution of Civil Engineers. Vol. XXVII.—1867-68.

The Indian Government, some years since, recognized the importance of this matter, and organized a *Forest* Department, somewhat on the basis of the Public Works Department, and already the good results of this policy are admitted by all those in India who have given any attention to the subject.

The following extract on the *Objects of Forest Management** are pertinent here.

"Forest management has two objects in view:—

"1. To produce and reproduce certain useful material.

"2. To sustain or possibly improve certain advantageous natural conditions.

"In the first case we treat the forest as a crop, which we harvest from the soil, take care to devote the land to repeated production of crops. As agriculture is practised for the purpose of producing food crops, so forestry is in the first place concerned in the production of wood crops, both attempting to create values from the soil.

"In the second case we add to the first conception of the forest [illegible] namely, that of a cover to [illegible] under certain conditions, and in certain [illegible] a very important relation to other conditions [illegible]

"[illegible] the forest growth [illegible] the soil and in re- [illegible] also in checking [illegible] evaporation, fur- [illegible] drainage and influenc- [illegible] which it is desirable [illegible] natural forest growth [illegible] influence is due [illegible] and to the mechan-

[illegible] the Division of Forestry, [illegible]

ical obstruction which the trunks and the litter of the forest floor offer.

"Any kind of tree growth would answer this purpose, and all the forest management necessary would be to simply abstain from interference and leave the ground to nature's kindly action.

"This was about the idea of the first advocates of forest protection in this country; keep out fire, keep out cattle, keep out the ax of man, and nothing more is needed to keep our mountains under forest cover forever.

"But would it be rational and would it be necessary to withdraw a large territory from human use in order to secure this beneficial influence? It would be, indeed, in many localities, if the advantages of keeping it under forest could not be secured simultaneously with the employment of the soil for useful production, but rational forest management secures both the advantages of favorable forest conditions and the reproduction of useful material. Not only is the rational cutting of the forest not antagonistic to favorable forest conditions, but in skillful hands the latter can be improved by the judicious use of the ax.

"In fact the demands of forest preservation on the mountains and the methods of forest management for profit in such localities are more or less harmonious; thus the absolute clearing of the forest on steep hillsides, which is apt to lead to dessication and washing of the soil, is equally detrimental to a profitable forest management, necessitating, as it does, replanting under difficulties.

"Forest preservation, then, does not, as seems to be imagined by many, exclude proper forest utilization, but, on the contrary, these may well go hand in hand, preserving forest conditions while securing valuable material; the first requirement only modifies the manner, in which the second is satisfied."

Article 61. Rainfall.

In considering the growth of any crop, the annual rainfall should not so much be taken into account as the particular portion of the rainfall that fell during the irrigating season, and its distribution during that time. In the majority of irrigation countries it was not the deficiency of rainfall throughout the year, but the fact that the rain fell at unsuitable times, that rendered irrigation essential. In some famine years in India, the aggregate of the rainfall throughout the year was more than ample to mature the crops, but it was almost useless for purposes of cultivation, as it fell at the wrong time.

The Honorable Alfred Deakin, M. P., of Victoria, states *:—

"The arid area of the United States, by the terms of Major Powell's definition, includes only lands where the rainfall is under 20 inches per annum. Over the great belt in which irrigation has so far had its chief development, the record for a series of years gives but little more than half that quantity, so that 10 to 12 inches may be taken as a fair average, though the extremes show a much wider variation. In Northern California, and among the mountains to the east, the rainfall rises to 40 inches, while in the deserts of Southern California it falls to four inches.

"In Western Kansas the fall, not infrequently, reaches 20 inches; but there, as with us, this is so irregular that the farmer who relies solely upon a natural supply loses more by the dry seasons than he can make in those which are more propitious. The question as to whether settlement increases the rainfall in the West, as it has increased it in the Mississippi Valley, is still undetermined; for, though popular

* Irrigation in Western America, Egypt and Italy.

opinion is decidedly in the affirmative, the State Engineer of Colorado points out that official records so far do not support the assertion. The exceptions to this are that Salt Lake, Utah, appears to be steadily gaining in depth, and that dew is now observed at Greeley, in Northern Colorado, a phenomenon quite unknown until irrigation had been practiced for some years. Nor does the mere amount of rainfall indicate sufficiently the necessity for an artificial supply of water, unless also the seasons in which it falls are taken into account. In parts of Dakota and Minnesota, where the rainfall only averages about 20 inches, dry farming is carried on; while in districts of Texas, where the figures are as high, it would be impossible to obtain the same results without irrigation. The explanation is that in Dakota nearly seventy-five per cent. of the rain falls in the season when the farmer needs it, as against about fifty per cent. in Texas. Indeed, a gradation may be observed in this scale from north to south, since in Kansas some sixty-five per cent. of the rain falls in the spring and summer, while in the extreme south, as at San Diego, only half of the whole rainfall, nine inches, falls in the spring, and is consequently useless for agriculture. There is some irrigation in Dakota, as also in Iowa and Wyoming, but not nearly so much as in the States to the southward, where, even if the rainfall were as high, its distribution would render it insufficient. A glance at the rainfall statistics of Victoria will show that, roughly speaking, one-half of it might be included in the arid area, or in that portion of the sub-humid area in which irrigation is little less essential.

"The valleys of the north and the great plains of the northwest, as well as the belt of level country immediately to the north and west of Port Phillip and the eastern coast of Gippsland, all feel the need of a regular

rainfall. Still, there is little of what would be called in America, desert land. The irrigated districts of Southern California are hotter and drier than any portion of our colony, resembling, indeed, the climate of Algiers, rather than that of Southern Europe. There it is always grass-less and almost rainless in many seasons, while in the country beyond Swan Hill, though the rainfall drops to ten inches and even less, there are still numerous seasons in which a fair crop of grass can be obtained.

In Victoria, the difficulty for the most part is, that the supply is sometimes insufficient, often irregular, or distributed so as to leave the crops unsupplied at a particular period. The critical season is generally that in which the crop is ripening, toward the end of spring and beginning of summer. A glance at our rainfall statistics for the last four years gives Horsham an average fall of about sixteen inches, and Kerong of about ten inches, of which at the first rather more, and at the second rather less, than twenty-five per cent. falls in the three months, September, October and November. If an emergency watering could always be obtained during this period, our northern farmers would be sure of a harvest, while, as it is, they run the risk of a complete failure every two or three years. So far as rainfall is concerned, then, Victoria appears to be in as good a position as any of the irrigated States except Western Kansas. Enough rain can be calculated upon to materially decrease the quantity of water required to be artificially supplied, and, in exceptional years, to render irrigation unessential. Though there have been, at long intervals, years in which this state of things has been reached in South-western America, yet they are so few as to but little affect the average. To make the comparison perfect, the fall in the various seasons in

Victoria would need to be tabulated for a number of years. The soil of its several districts would also have to be carefully analyzed, for it is to be remembered that one lesson of American experience is that soils which to the 'dry farmer' gave but faint promise of any productiveness, have proved extremely fertile when exposed to frequent saturation and continuous cultivation. The quantity of water needed is also affected by temperature, for the higher it reaches the more water is demanded. The loss by evaporation has not yet been determined for the several States, but, it is stated, that in very arid tracts, it rises to over sixty inches per annum.

As favored in rainfall as America, Victoria is less favored than India, Italy or France, where the precipitation is often twice as great. The fact that irrigation is resorted to under such conditions should be borne in mind, when we consider the wisdom of securing an artificial supply in places where the yearly fall is often sufficient."

The Statistical Review of the Irrigation Works of India for 1887–88, has the following on rainfall:—

"It has not infrequently been assumed that the probabilities of the success of a new irrigation project can be gauged by a consideration of the incidence of the rainfall on the tract commended. The rainfall is, no doubt, one of the chief factors to be considered, but the statistics of rainfall* show conclusively that there must be other factors of at least equal, if not greater, weight, which must be taken into account in determining the success or failure of an irrigation system. For example, the rainfall in Bombay is generally scanty, while at Madras it is copious, but in the former case the irrigation works are entirely unremunerative, whereas in Madras they are, with one exception, most lucrative.

* Not given in this work.

"Even a more striking instance can be found in Madras itself. The Cauvery Canals, which irrigate a larger area and pay a far higher percentage on capital than any other system in India, lie in a district where the average rainfall is 53.9 inches in the year; whilst the Kurnool Canal, which is the most conspicuous failure of all irrigation works in India, lies within 300 miles of the Cauvery, in the same Province, in a tract where the average rainfall is 28.9 inches, or only slightly more than half that which falls on the Cauvery Canals. The causes which produce these striking differences are but little understood, and the available statistics afford no clue to them. It may, however, be said that in Madras the temperature is generally so equable that it is possible to grow two, and even three, crops of rice on the same field during the year; this is not possible in the more variable climate of Bengal, where the total rainfall is not greatly different from that of Madras. It may be that this climatic difference explains the great discrepancy in the results obtained in the two Provinces. It should also be noted that the actual amount of the rainfall is of less importance than its distribution. Differences in soil and in methods of cultivation have also great weight in determining the success of an irrigation project. * * * * Whatever the causes may be which should determine the results obtained, it must be admitted that much ignorance has prevailed concerning them, and this has led to the construction of many works which have signally failed to produce the results which were anticipated by their projectors."

The following extract is taken from *Engineering News* of May 11, 1889:—

"There is no part of California where the people are more in earnest about irrigation than in Colusa County.

California (see page 175), where they have an annual rainfall of 30 inches. There are two classes of lands requiring irrigation here—one, lands which will yield crops without irrigation, but which will double their yield under the influence of a regular supply of water—say a cubic foot per second to 150 acres—during the growing season; the other, desert lands, which will yield nothing at all without an artificial supply of water, either from a system of irrigation works or artesian wells."

The following table is from a paper by Mr. P. O'Meara, M. Inst. C. E., in the Transactions of the Institution of Civil Engineers, Vol. LXXIII:—

Article 62. Evaporation.

In countries where irrigation is conducted on an extensive scale, the evaporation, that is, the depth of water evaporated annually, does not materially differ. The records of experiments given below in America, Italy, France, Spain, India and Egypt, prove this.

The records of evaporation published by the State Engineering Department of California, show that the mean annual evaporation at Kingsburg bridge, Tulare County, California, for the four years from 1881 to 1885 was 3.85 feet in depth, when the pan was in the river, which is equal to an average depth of one-eighth of an inch per day for a whole year.

For the same period the evaporation, when the pan was in air, was 4.96 feet in depth, that is, equal to a mean daily depth of evaporation throughout the year, of less than three-sixteenths of an inch per day.

The greatest evaporation was in the month of August, when it was more than one-sixth of the evaporation for the whole year. The average for this month is one-third of an inch per day.

During the months when the largest quantity of water is used for irrigation in this district, the table shows that the mean evaporation was:—

For March one-twelfth of an inch per day.

For April one-twelfth of an inch per day.

For May one-fifth of an inch per day.

Mr. Walter H. Graves, C. E., states:—*

"Evaporation is very nearly a constant quantity.

* * * * * * * *

Observation and experiment by the writer in various parts of Colorado tend to show that evaporation ranges

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* Irrigation and Agricultural Engineering in Transactions of the Denver Society of Engineers—1886.

from .088 to .16 of an inch per day, during the irrigating season."

To some people these depths of evaporation may appear very small. Let us, therefore, examine the result of observations in other countries:—

Colonel Baird Smith, in his work on Italian Irrigation states that, in the north of Italy and center of France, the daily evaporation varies from one-twelfth to one-ninth of an inch per day; while in the south and under the influence of hot winds it increases to between one-sixth and one-fifth of an inch per day.

In 1867 the total evaporation in Madrid, Spain,* was sixty-five inches in depth. In July of the same year according to the returns of the Royal Observatory, it was 13½ inches in depth or less than half an inch per day, and in May of the same year it was only one-quarter of an inch per day. July was the hottest month in 1867, and it was estimated that during this month the total evaporation of the Henares Canal, carrying 105 cubic feet per second, or 5,250 miner's inches, under a four inch head, amounted to only three-fourths of one per cent. of the total flow.

W. W. Culcheth, C. E.,† states as the result of his investigation on the Ganges Canal in Northern India, that for evaporation, one-quarter of an inch per day over the wetted surface may be taken as the average loss from a canal.

Dr. Murray Thompson's ‡ experiments in the hot season in Northern India, with a decidedly hot wind blowing, gave an average result of half an inch in depth evaporated in twenty-four hours.

* Irrigation in Spain, by George Higgin, M. Inst. C. E., in Transactions of the Institution of Civil Engineers. Volume XXVII.—1867-68.

† Transactions of the Institution of Civil Engineers. Volume LXXIX.

‡ Professional Papers on Indian Engineering. Vol. V. Second Series.

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‡ Professional Papers on Indian Engineering. Vol. V. Second Series.

In Hyderahad in the Deccan, in India, it was found that the mean evaporation from a tank or reservoir was 0.165 inch per day.

In Nagpur, in India,* the total depth evaporated from October, 1872, to June, 1873, was four feet, which, distributed over the period of the experiment, 242 days, gives an average depth of .0165 feet, or 0.198 inch, being about one-fifth of an inch per day.

Colonel Fyfe, R E.,† states that in large reservoirs in India, about two square miles in area, the amount of evaporation, that he made allowance for was about three feet in depth per annum in the Deccan, and something less in the Concan district in India.

Major Allan Cunningham, R. E.,‡ conducted experiments, lasting twenty-five months, from 1876–79, to measure the evaporation from the Ganges Canal. He states that, the most remarkable feature of the results is their extreme smallness, amounting to only about one-tenth of an inch per day on the average near Roorkee; whereas one-half inch per day is said to be a common rate in India for evaporation on land. This led at first to the suspicion of the introduction of water from without; but after considering the possible sources of this, namely, leakage, spray, rain, dew, wilful tampering, it still seems that the results may be accepted as substantially correct. The real cause of the small evaporation appears to be the unusual coldness of the canal water, for instance, on May 22, 1877, at 2:30 P. M., the temperature of the air was 165° in the sun, and 105° in the shade, whilst that of the water was only 66° inside the

* The Nagpur Waterworks, by A. R. Binnie, M. Inst. C. E., in Transactions Institution of Civil Engineers. Vol. XXXIX.

† Transactions Institution of Civil Engineers. Vol. XXXIX.—1874-75.

‡ Recent Hydraulic Experiments in Transactions of the Institution of Civil Engineers. Vol. LXXI.—1883.

pan and 65° in the canal; also the highest recorded temperature of the canal water was only 75½°. The canal, in fact, takes its supply from the Ganges, a snow-fed river, at its exit from the hills.

It was, indeed, found that the canal evaporation increased with distance from the head of the canal at Hurdwar on the Ganges. Thus, out of the forty results, twenty-eight were taken near Roorkee, and twelve near Kamhera, at distances of eighteen and fifty-two and one-half miles from the head-works; the evaporation at the latter was much the larger, comparing, of course, similar seasons, being about 0.15 inch against 0.10 inch on an average. This is, no doubt, due to the gradual heating of the water under the hot sun, with increased distance from the head.

Taking the Roorkee estimate of one-tenth of an inch per day, the total evaporation from the whole surface of the canal and its branches, about 487,000,000 square feet, amounts to about forty-seven cubic feet per second, which is about $\frac{1}{150}$ part of the full supply of the canal, or in other words, ten minutes full supply daily.

Little connection could be traced between the evaporation and the meteorological elements; the temperature of the water, which depends chiefly on the amount of snow water in the Ganges, being probably the governing element.

M. Lemairesse's* observations at Pondicherry, in French India, give a daily evaporation of from three-tenths to half an inch in depth per day.

Trautwine made observations in the Tropics and he found the evaporation from ponds of pure water to be at the rate of one-eighth of an inch per day, but he

* The Irrigation of French India in Professional Papers on Indian Engineering, Volume I. Second Series.

observes that the air in that region is highly charged with moisture.

Mr. Willcocks, C. E., in his work on *Egyptian Irrigation*, states that Linant Pasha considered the evaporation in Upper Egypt, as about equal to one-third of an inch per day throughout the year. As a result of his own observations, Mr. Willcocks gives the evaporation for one year in Upper Egypt, as equal to six feet in depth, and in Lower Egypt, as equal to 2.4 feet in depth.

The following table given by General Scott Moncrieff, R. E., shows the general conditions of temperature and rainfall, as measured at Orange, eighteen miles north of Avignon, and at Marseilles in France, for periods of thirty and twenty years respectively:—

TABLE 20. Giving temperature and rainfall in the South of France.

	Mean Temperature.			Greatest heat in degrees. F.	Greatest cold in degrees. F.	Annual rainfall in inches.	No. of wet days in the year.
	Summer	Winter.	Whole y'r.				
Orange ...	71	41	56	104	5	26.6	96
Marseilles.	70	45	59	87	22	12.8	59

This table shows, in a striking way, the modifying influence which the sea has over climate, the extreme range at Marseilles being only 65°, while at Orange, ninety-three miles distant, it is 99°. The annual rainfall, scanty as it is, does not fully denote the extent to which this part of France suffers from drought, for at Avignon it often happens that there is not a shower of rain during the three hottest months of June, July and August. The evaporation in the plains of Languedoc, not far distant, has been estimated at .079 inch per diem, and it is probably about the same in Provence.

Article 63. Percolation.

Taking a broad view of percolation in channels and reservoirs, through their beds and banks, it denotes infiltration, seepage, absorption and even leakage. If the leakage is of a large quantity through a bank of earth, that bank is not likely to last long.

"In every new canal, through sandy loam, the loss by percolation at first is very serious. Gradually the ground gets saturated, and at the same time the interstices of the porous material of the bed and banks get filled up with particles of clay, which diminish the percolation. The bed of a canal acts as an *elongated filter*, It is well known that the sand of a water works filter bed, if not periodically washed, or if not replaced with clean sand, the interstices between the particles of sand get filled with silt, and the filter ceases to act, or acts so slowly as to be practically useless. The same thing takes place in a canal, but at a slower rate than in a filter bed. There is less deposit in an irrigation canal, in the same time, than in a filter bed, as the greater part of the finer particles of silt do not deposit in it, but are carried in suspension until the water reaches the land to be irrigated."*

Mr. Walter H. Graves, C. E., in a paper read before the Society of Engineers in Denver, Colorado, in 1866, states:—

"The factor of seepage is a variable one, depending mostly upon the nature of the soil, and gradually grows less through a long term of years. Evaporation is very nearly a constant quantity, depending on the altitude of the locality and the prevailing meteorological conditions. In calculating for the loss from these sources, evapora-

*Report on the proposed Works of the Tulare Irrigation District, California, by P. J. Flynn, C. E.

tion and seepage, in the older canals about twelve per cent. should be deducted from the carrying capacity."

Mr. P. O'Meara, in a paper in the Minutes of Proceedings of Inst. C. E. for 1883, states:—

"From a short time after irrigation is established in any district, the quantity of water required will gradually become less, till an equilibrium is established between the amount of water supplied in the irrigating season, and the quantity removed by filtration and evaporation. * * * * * *

"It is a question whether, supposing irrigation were carried out to its full extent in Colorado, there would be any loss of irrigating power other than that due to evaporation. Losses occurring through absorption and surface flow are not final. The waters absorbed or wasted reappear, probably with undiminished volume, lower down in the streams."

Mr. Boyd, President of the State Board of Agriculture of Colorado, in a communication to the Institution of Civil Engineers in 1883, states:—

"In a volume of six cubic feet per second of water flowing in a lateral two miles in length, not less than one-tenth would be lost by soaking and evaporation."

Mr. E. B. Dorsey, C. E., in a paper in the Transactions of the Am. Soc. C. E., Volume 16, 1887, states:—

"That in some of the Colorado canals the loss from evaporation and seepage is estimated at fifty per cent., which is excessive, and shows that the canal is constructed in bad soil, or that there must be something the matter with the construction. Twenty per cent. ought to be, under ordinary circumstances, a liberal loss from these causes, and this should largely diminish as the banks and bottom of the canal become compact."

Mr. George G. Anderson, C. E.,* made measurements

* Transactions of the American Society of Civil Engineers. Volume XVI, 1887.

on the High Line Canal, in Colorado, in the middle of July, 1886, and found that where 156 cubic feet per second were passing into the head-gates only 80 cubic feet per second were passing a point 45 miles from the head-gates, and no water was used for any purpose in the intermediate distance. This was during the very hottest and driest period of an unusually hot and dry summer in Colorado. The soil through which this canal passes is in many places very pervious. There are long stretches of fine sand, and in places the canal bottom is in rock badly fissured. The alignment of the canal is very crooked, and, no doubt, a great loss is experienced from this source. It is to be expected that this serious loss will gradually diminish as the canal bed and sides become compact and puddle naturally. But to estimate a smaller loss from these causes than twenty-five per cent. would scarcely be wise."

In a paper by Mr. C. Greaves,* he showed that the percolation through ordinary soil, as compared with sand, was only about one-third, whereas the evaporation from a surface of ordinary soil was four times that from a surface of sand.

Mr. G. F. Ritso, C. E., in his description of the irrigation of the Canterbury Plains, New Zealand, states:—†

"Of the canals in alluvial soil that the percolation is small, as the constant tendency of channels is to silt up and to become more water-tight."

The Martesana Canal, in Italy, of a capacity of 981 cubic feet per second and twenty-eight miles in length, was estimated to have lost from evaporation, seepage, and illegal abstraction, 3.75 cubic feet per second per mile of canal. We have here, however, an additional source of loss, that by illegal abstraction.

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* Transactions of the Institution of Civil Engineers. Vol. XLV, 1876.

† Transactions of the Institution of Civil Engineers. Vol. LXXIV, 1883.

Engineers in Lombardy calculate the absorption in each watering, of about four inches in depth, as ranging from one-third to one-half of the total quantity of water employed. This is when the general period of rotation is about fourteen days. From observation, it has been concluded that the balance of the water reaches channels at a lower level, and is again available for the irrigation of lower lands.

On the Marseilles Canal, in France, the losses by percolation, evaporation, and at the settling or silt basins, was estimated at 58 cubic feet per second, or sixteen per cent. of the full supply of 353 cubic feet per second.

Ribera estimated the total loss from evaporation and percolation on the Isabella Canal, in Spain, a masonry-lined channel, at two per cent.

Nadault de Buffon gives the average percentage of loss on canals from evaporation and percolation at 15 per cent. of the total volume carried. He does not, however, mention under what circumstances such a percentage may be expected.

In a project for a canal from the Rhone, in France, of over 2,000 cubic feet per second, it was calculated that one-sixth would be lost by evaporation, percolation, etc.

In designing the Agra Canal, India, the loss by absorption and percolation was estimated at 0.23 cubic feet per 100 cubic feet per mile of canal.

After the completion of the Ashti Tank, in India, observations were made, and it was found that, out of a supply of 1,348,192,450 cubic feet, the loss from evaporation, percolation and seepage through the subsoil of the tank combined, amounted, in a year, to 233,220,240 cubic feet, or about 18 per cent. of the supply.

In the Irrigation Revenue Report of Bombay for 1889–90, it is stated, as the result of gaugings, that the

Ashti Tank lost during the year by evaporation, absorption, etc, 7.08 feet in depth of water over its mean area.

In the same Report the result of some experiments in the loss of water in small canals was given:—

"On the Palkhed Canal, 14.87 miles in length, there was a loss by leakage and evaporation of 0.44 cubic feet per second per mile, or nearly forty-eight per cent. of the supply.

"Before taking observations for leakage experiments all the irrigating outlets were closed; gaugings were made with ordinary floats and were read in each mile, which gave the average result of loss as forty-eight per cent. The only feasible way of reducing the leakage seems to be to keep the canals clear of silt and weeds. No other precautions seem practicable, unless in the way of managing all outlets better, escapes included.

"On the Ojhar Támbat Canal (1.75 miles in length) there was a loss of 0.49 cubic foot per second, or nearly thirty per cent. of the supply."

The percentage of loss on the above canals is very great, and it is seldom that the loss is so great in a channel carrying silt, and that has been in use for some years.

When the Ganges Canal was flowing at least 6,000 cubic feet per second, during October, 1868, a year of drought, Colonel H. A. Brownlow, R. E., estimated the loss by absorption at twenty per cent. He states that the estimate of loss by absorption (twenty per cent.) may be considered somewhat low for a year of drought, but that the long continued high supply in the canal must, after some time, have checked, in a great measure, the drain upon itself by fully saturating the adjacent ground. In fact, the greater ease with which the gauges were kept up during October and November, as compared with August and September, was a matter of common remark at the time.

In the original design of the Ganges Canal its discharge, at full supply, was fixed at 6,750 cubic feet per second. Of this quantity, it was assumed that 1,000 cubic feet per second would be lost by evaporation, absorption and navigation, and that the remainder would be available for irrigation.

Mr. J. S. Beresford, C. E., states as the result of Indian experience, that old canals give higher duties of water than new canals, or, in other words, that there is less loss of water through the material forming the channel in old than in new canals.

Sir B. Baker, C. E., has stated:—*

"In a porous soil like that of Egpyt, it was impossible to confine water simply by raising the bank, because it would find its way by percolation underneath, and it came up to the surface and washed the salt out and killed vegetation. He had ascertained that the water percolated at the rate of about one mile from the river in a week. That is to say, the water in a well one mile from the river would begin to rise about a week after the water in the river had begun to rise. It would be seen that that was an exceedingly important matter as affecting many questions of drainage in London. If the tide were not of twelve hours but a week's interval, the greater part of the low-lying districts in London would be much injured by the percolation of tidal water; but at present it did not follow up quickly enough to exert a destructive hydrostatic pressure upon the thin basement walls of houses near the river."

* Transactions of the Institution of Civil Engineers. Vol. LXXIII, 1883.

Article 64. Drainage.

As a rule, the drainage of irrigated land will take care of itself, if the natural drainage channels are left free and unobstructed. If it is found that, before irrigation is introduced into a district, the country is flooded and water-logged after rains, then it is likely to be in a worse condition after the land is irrigated, and drainage will be absolutely necessary for the success of the irrigation and the health of the district.

In many cases in this country, although irrigation dates back but a few years, the natural drainage outlets have been converted into irrigation channels, with the very worst results. In this way, while the supply to be drained off had been increased in quantity, the drainage channels have been diminished in carrying capacity.

If the subsoil and surface water cannot escape freely by the natural channels, super-saturation follows, and the ground becomes water-logged. Stagnant water is very injurious to crops, and it generates disease and pestilence. Many irrigation districts in this country show the evil effects of too much irrigation combined with defective drainage. One of the least evils is a dense and troublesome growth of weeds, and as a consequence waste land. The cultivator suffers in health and pocket.

To construct irrigation canals without efficient surface drainage, and, as has sometimes been the case, to obstruct the natural drainage of the country, by the mproper location of canals, without making adequate provision for allowing the surface drainage to pass away, tend to the certain formation by artificial means, of those evils that exist in the neighborhood of natural swamps, and hence, the importance of paying every attention in the preparation of projects, and the construction of works,

with the view of avoiding those defects which, if permitted in the first instance, will certainly have to be remedied at some future time, at considerable cost, both direct and indirect. Whatever excuses may have been admissible in past years, when the science of constructing irrigation works was less understood than it is at the present day, no justification can now be pleaded for the repetition of similar errors.

On the reconstruction of the Western Jumna Canal in 1820, after a suspension of its usefulness for more than half a century, the original mistake of a bad location was repeated. Instead of being carried along the watershed lines it was taken through the drainage of the country, by interfering with which, serious consequences resulted in the creation of swamps and the occasional submergence of lands which might, by a proper location, have been brought under cultivation. But besides rendering lands uncultivatable, and so curtailing the extent of area capable of growing for a poor and highly taxed people, the healthfulness of the neighborhood of these swamps became seriously impaired, and the population was found to be on the decrease in the vicinity. In some cases land became waterlogged, and therefore useless, for cultivation, whilst in others it became covered with a peculiar saline efflorescence, known as alkali in America, and *reh* in India. After investigating the above state of affairs, the Indian Government adopted measures to abate the evils of the defective irrigation.

Egypt is now suffering from the super-saturation of its land and want of proper drainage. Mr. W. Willcocks, Asso. Inst. C. E., states:—*

"The canals are so disproportionately large during flood, that they send down into the lower lands further

* Irrigation in Lower Egypt in Transactions of the Institution of Civil Engineers. Vol. LXXXVIII.—1886-87.

north such an excessive volume of water, that all the canals, escapes, and drainage cuts are full to overflowing with flood water, and are in consequence unable to perform their proper functions. The country during flood is divided into a number of islands surrounded by water at a high level. The natural consequence is that salt efflorescence is greatly on the increase in the lands under cultivation."

Again he states:—

"The conversion of all the drainage cuts into irrigation canals, was all that was needed to destroy the higher lands. This soon followed."

There are several districts in California where a few years since the great want was water, but where, at the present time, the pressing want is drainage. A small percentage of the quantity of water required a few years since to irrigate a certain area. is now sufficient to insure a crop, as the sub-soil is so saturated with water, that very little flooding is now required in comparison with the first few years after the introduction of irrigation.

The same thing has happened in Colorado. Mr. G. G. Anderson, C. E., states:—*

"In Colorado, as in most other irrigation countries, the necessity of carrying on drainage and irrigation simultaneously is being impressed upon practical men more and more every year. Although it is a rare occurrence when these works are successfully conducted together, it is regrettable to note the large and yearly increasing area of low-lying lands going to waste, and which are during the irrigating season stagnant swamps breeding disease. The frequency of typhoid fever and other epidemics in the fall of the year, is doubtless due

* The Construction, Maintenance and Operation of Large Irrigation Canals in Transactions of the Denver Society of Civil Engineers and Architects, Vol. I.

to this cause, so that, from a sanitary point of view at least, drainage must be speedily undertaken."

To avoid this defective irrigation, some means should be adopted in irrigation districts, to prevent the use of the natural drainage channels, for any purpose whatever, but that of conveying away the drainage water that reaches them.

A good effect will be produced by restoring to their natural state such drainage outlets as have been converted into irrigation channels, and, if required, their carrying capacity can be increased by widening and deepening them and taking out the sharp bends. An annual clearance of debris, brush and weeds will have a good effect in keeping up their discharging capacity.

A great deal has been written, usually by mere theorists, on subsoil drainage in connection with irrigation.

In an able paper by Mr. H. Scougall, C. E.,* he states:—

"Now, to prevent the appearance of alkali on our lands, water must be used sparingly for irrigation purposes, and not a drop more than is actually necessary to promote the growth of our crops should be poured on the land."

This is quite right and to the point. Again he states:—

"No good system of irrigation should be without drainage; that is, drains some 18 or 36 inches below the surface which will carry off all surplus water."

Whilst it is a fact that no perfect system of irrigation should be without subsoil drainage, still it is a hard fact, that no country in the world requiring irrigation, can at the present moment pay for such a system as is indicated by Mr. Scougall and at the same time pay for an irrigation system. Doubtless, exceptionally small areas

* The Construction of Canals for Irrigation Purposes read before the Polytecthnic Society of Utah, March, 1891.

can be pointed out, having the two systems in operation, but what we refer to is a combined system covering a large area such as is commanded by the Agra Canal in India, or the Calloway Canal in California.

To show the immense magnitude of such work if applied to the irrigation districts of India, the following extracts are taken from the Statistical Review of the Irrigation Works of India, 1887–88:—

At the end of the financial year, 1887–88, there were completed in India 5,520 miles of main canals and 17,155 miles of distributaries, and these works irrigated over 10,000,000 acres. This includes only the great works. The Minor works irrigated 2,000,000 acres more. There were, therefore, over 12,000,000 acres of land irrigated in 1887–88. The subsoil drainage of this area of land could not be carried out, to a successful completion, by any country in the world, that is, as a paying investment.

For large districts the subsoil drainage would cost much more than any irrigation system by open earthen channels. The cost at present prohibits the use of subsoil drainage on an extensive scale.

If all the drainage channels are improved to their outfall into some river, and new open drainage cuts made where required, then this will, as a rule, prevent surface flooding and super-saturation of the soil, and this is as much as can be done under the present financial condition of irrigated countries.

Article 65. Defective Irrigation—Alkali—The Effect of Irrigation on Health.

The chief objections urged against irrigation are the unhealthfulness that follows the super-saturation of the soil, and the injury to the land caused by alkali, known in india as "reh." These two evils can, in a great measure, be avoided by using only just sufficient water to mature the crop, but not enough to saturate the whole sub-soil.

The returns of the duty of water in America, go to prove that, as a rule, too much water is used. India, Egypt and America are suffering from alkali in the land, and the evil is on the increase.

Engineering News of February 26th, 1887, contains the following paragraph:—

"Professor Hilgard, of the State University of California, warns the people of the Pacific Coast that land irrigation may be overdone. He says that more attention must be paid to under-drainage, and sustains his arguments by existing conditions in the irrigated plains of Fresno, Tulare and Kern, where there was formerly no moisture within thirty or forty feet of the surface, while water now is found almost anywhere within three to five feet. The roots of trees and vines have been forced to the surface and the alkali accumulating through centuries is also brought upward. He recommends as a remedy, laws providing for proper location and construction of the ditches."

Where water is available frequent washing of the surface of alkali land will do much to reclaim it. The land should be flooded to a depth of a few inches, and left in this condition for a few days, then drawn off, and again flooded with fresh water, and this operation should be repeated until the surface of the land is cleared of alkali.

Opinions as to the effect of irrigation on health are somewhat conflicting, and for this reason we give below opinions from different sources on this subject.

Dr. H. S. Orme, Member of the State Board of Health of California, states, with reference to the influence of Irrigation on Health *:—

"The effect of the irrigation of the agricultural lands, particularly in California, upon public health is one of growing importance, and inasmuch as the available evidence bearing upon the subject is somewhat contradictory, it is necessary to note the conditions of locality, with respect to soil, temperature, humidity and drainage, wherever irrigation is practiced.

"Although irrigation has been carried on in California since the first establishment of the early missions by the Franciscan Fathers, more than a century ago, very little progress has been made in the scientific application of the system, the object of the cultivator being apparently only to get the water upon his land, without regard to the method employed.

The application of the water used in irrigation varies greatly in manner, but may be described as two different methods, viz: first, by flooding the whole surface of the land from open ditches (*Zanjas*); and second, by sub-irrigation, that is a conveyance of the water through pipes beneath the surface of the ground, which have openings at intervals, protected by upright pipes.

So far as the effect on health is concerned the latter method will not be considered, because of the very limited extent to which sub-irrigation is being applied.

In the case of the application of water by *flooding* the land from open ditches, the various reports made by impartial authorities, are, in some respects, conflicting.

* Appendix to the Eighth Biennial Report of the State Board of Health, California.

For instance, in Los Angeles, Ventura, Santa Barbara, San Bernardino and San Diego counties, where irrigation has been carried on for over a hundred years, the testimony is strong to the point that, there is no striking difference in the amount of malarial diseases, whether irrigation is practised or not. On the other hand, if we consult the records of some other portions of California, we find an increase of malarial fevers with the increase of irrigation, too intimately connected to be overlooked. The reasons for this are not difficult to discover. In Los Angeles and other valleys in extreme Southern California, where the soil is, as a rule, sandy or gravelly loam of unknown depth, the water in irrigation either sinks into the ground, or, if there is much surface slope, immediately drains at, or near, to the surface. In such sections of country there is great freedom from malarial diseases. Along the bottom lands of rivers where the slope is insufficient to insure good drainage, or where the soil is constantly saturated, the case is different. Here there is more or less intermittent and remittent fever during the warmer season of the year. In the case of swamp or overflowed lands, especially those having a heavy adobe soil, as well as those which remain wet and boggy from the winter rains, and are in summer kept in a saturated condition by artificial means, containing also an excess of decomposing vegetable matter and many stagnant pools, malarial diseases of the most pronounced type are very prevalent. In such localities all zymotic diseases are much worse in summer than in winter, a consequence which naturally results from the high temperature and increased evaporation. The fact that the people, living in these low, wet adobe sections of country, are dependent upon impure or surface water for drinking and domestic purposes, greatly aggravates the difficulty. In-

deed, it has been more than once demonstrated that people living in a "fever and ague" country are tolerably exempt from the fever if they drink only pure water.

In referring to defective irrigation in India, the *Engineer*, London, of June 23, 1871, has the following:—

"It is notorious that wherever irrigation is carried on, cruel malarious diseases as surely follow, and unless Dr. Cutliffe's report, in 1869, 'On the Sanitary Condition of the lands watered by the Ganges and Jumna Canals' very greatly errs, it is very questionable whether the aggregate increased mortality in a number of years, due to irrigation, does not even exceed what that of a periodic famine would be.

"There are very extensive portions of the irrigated districts where subsoil drainage would not only be practicable but easy, and would entirely remedy many of the existing evils distinctly traceable to over irrigation.

"Nothing beyond an extension of surface drainage appears even yet to be contemplated; but until such works are regarded as merely the basis of subsoil drainage to follow, we can look for little real improvement in the system of agriculture in India."

India is not able to pay now, and it is not likely that she will ever be able to pay, for a system of subsoil drainage. (See Article 64.)

On the subject of defective irrigation, we have more recent information, which is herewith given in the testimony of Dr. W. W. Hunter, who has had long experience in India:—*

"Even irrigation itself occasionally displaced a population, and, in several parts of India, created a safeguard against dearth only at the cost of desolating the villages by malaria."

* Life of Lord Mayo, page 326, Vol. 2.

We have additional information on the same subject relative to Europe, given by Mr. G. J. Burke, M. Inst. C. E.,* who had a large experience on Irrigation Works in India: He was of the opinion that:—

"Drainage and irrigation ought to go together; but how many engineers had seen both drainage and irrigation properly carried out at the same time? He certainly never had. He had seen many of the irrigated districts in Europe, and nearly all in India, and the result of his experience was, that in the irrigating season, when the canals were full, the low-lying lands became swamps, generating disease and pestilence; and he had no doubt that a good deal of unhealthiness, in countries where canal-irrigation was extensively practised, was owing to the neglect of drainage to carry off the surplus water."

Article 66. Cost of Irrigation per acre in different countries.

In America, as a rule, the land and water go together, and the only expense the landowner is subject to is, that of maintenance of the Canal.

In India, on the contrary, the Government owns all the great canals and sells the water to the cultivators.

In the Statistical Review of the Irrigation of India, 1887–88, it is stated that the rates which are charged for the use of water for irrigation vary very largely in different parts of India and for different crops. In some cases a charge is made for a single watering, and in others a special rate is taken for water used during certain months, but generally the charge is an average rate for irrigating the crop to maturity. Excluding very excep-

*Transactions of the Institution of Civil Engineers. Vol. LXXIII, 1883.

tional cases, it may be said that this rate varies from forty cents an acre for rice crops in some parts of Bengal and Sind, up to eight dollars, which is not an extreme rate in Bombay for sugar cane crops. The average rate is less than $1.20 an acre. (The rupee is here assumed as equal to forty cents.)

In the Punjab Revenue Report on Irrigation for 1889-90, it is stated that the average water rate for this year, for the Western Jumna Canal was about one dollar per acre.

TABLE 21. Giving cost of irrigation per acre in different countries.

CANAL OR LOCALITY.	COUNTRY.	Rate per acre in dollars.	AUTHORITY.
Ganges Canal	India	$1 12	F. C. Danvers, C. E. Trans. I. C. E., vol. 33.
Eastern Jumna Canal	India	1 16	F. C. Danvers, C. E. Trans. I. C. E., vol. 33.
Western Jumna Canal	India	1 20	F. C. Danvers, C. E. Trans. I. C. E., vol. 33.
Baree Doab Canal	India	1 17	F. C. Danvers, C. E. Trans. I. C. E., vol. 33.
India (Rice)	India	2 50	G. J. Burke, C. E. Trans. I. C. E., vol. 73.
Madras	India	3 00	J. B. Morse, C. E. Trans. I. C. E., vol. 73.
North West Provinces	India	1 25	J. B. Morse, C. E. Trans. I. C. E., vol. 73.
Soonkasela Canal	India	3 00	J. H. Latham, C. E. Trans. I. C. E., vol 34.
Ceylon	Ceylon	50	J. B. Morse, C. E. Trans. I. C E., vol. 73.
Lower Egypt	Egypt	5 00	Gen. Scott Moncrieff—19th century—Feb., 1885.
Alpines Canal	France	$2 to $3	George Wilson, C. E. Trans. I. C. E., vol. 101.
Canal from Rhone	France	10.00	Engineering, 29 June, 1877.
Marseilles Canal	France	6 50	George Wilson, C. E. Trans. I. C. E., vol. 51.
Verdon Canal	France	5 50	George Wilson, C. E. Trans. I. C. E., vol. 51.
Henares Canal	Spain	7 25	George Wilson, C. E. Trans. I. C. E., vol. 51.
Esla Canal	Spain	5 75	George Wilson, C. E. Trans. I. C. E., vol. 51.
Colorado	America	$1 50 to $3	R. J. Hinton—Irrigation in the United States.
Truckee Valley, Nevada	America	5 00	Quoted by A. D. Foote, C. E.

Article 67. Annual earning of a cubic foot of water per second.

The following extract is taken from a work by the Honorable Alfred Deakin, M. P., of Victoria.*

"At Los Angeles, California, water is sold by what is called a "head," which under their loose measurement, varies from two cubic feet to four cubic feet per second, at $2 per day or $1.50 per night in summer within the city, twice that price outside of its boundaries, and half the price in winter. At Orange, Southern California, and its neighboring settlements, the price for a flow of about two cubic feet per second is $2.50 for twenty-four hours or $1.50 per day and $1 per night, and in winter $1.50 for twenty-four hours. At Riverside the cost is about $1.90 per day or $1.25 per night, for a cubic foot per second, or $3 for the twenty-four hours. These prices varying indefinitely as the conditions of sale vary, furnish but an insecure basis for any generalization. Possibly a better idea of the importance of water, than can be derived from any list of purchases and rentals in particular places, may be obtained by a glance at its capital value. It has been calculated that the flow of a cubic foot per second for the irrigating season of all future years is worth from $75 to $125 per acre in grain or grazing country, to $150 in fruit lands. This is the price paid to apply such a stream to a special piece of land for as long as the farmer may think necessary, the knowledge that an excess of water will ruin his crop being the only limit. But if a flow of a cubic foot per second were brought in perpetuity without any limit to the acreage to which it might be applied, or the time or circumstances of applying it, the capital value of such a stream in Southern California to-day would be at least $40,000.

* Irrigation in Western America, Egypt and Italy.

The following table is compiled from various sources:

TABLE 22. Showing the annual earning of a cubic foot per second in different countries.

NAME OF CANAL.		Annual earning of a cubic foot per second.	AUTHORITIES.
Ganges,	1866-67...	$187	Russel Aitken, C. E. Trans. I. C. E., 1871-2.
Ganges,	1867-68...	195	Russel Aitken, C. E. Trans. I. C. E., 1871-2.
Ganges,	1868-69...	262	Russel Aitken, C. E. Trans. I. C. E., 1871-2 (year of drought).
Eastern Jumna,	1866-67...	261	Russel Aitken, C. E. Trans. I. C. E., 1871-2 (year of drought).
Eastern Jumna,	1867-68...	260	Russel Aitken, C. E. Trans. I. C. E., 1871-2 (year of drought).
Eastern Jumna,	1868-69...	326	(Year of drought.)
Western Jumna..................		249	F. C. Danvers, C. E. (year of drought).
Baree Doab..................		164	F. C Danvers, C. E. (year of drought).
Ganges,	1870-71...	220	Col. W H. Greathead. Trans. I. C. E., vol. 35.
Eastern Jumna,	1870-71...	295	Col. W H. Greathead. Trans. I. C. E., vol. 35.
Piedmont		80	Colonel Baird Smith.
Lombardy..................		75	Colonel Baird Smith.
Elche and Lorca, Spain..........		11000	George Higgin, C. E., in Trans. I. C. E., vol. 27.
Henares, Spain..................		1875	George Higgin, C. E., in Trans. I. C. E., vol. 27.
Canals in Colorado..............		33	H. M. Wilson, C. E., in Trans. Am. Soc. C. E., 1890.

Article 68. Cost of Canals per Acre Irrigated and per cubic foot per second.

The following table is taken from the most reliable sources available, but no doubt there are errors in it as the account of cost varies by different authorities. It is merely given to show approximately the cost of irrigation canal work in different countries. It is almost impossible to make anything like an accurate comparison of the cost of works in different countries, there are so many different matters entering into the subject. For example, the Ganges Canal is estimated to have cost $2,487, per cubic foot per second, whilst the Orissa canals are stated to have cost only $1,000. The former canal, however, has a greater number per mile of expensive works, such as bridges, falls, regulators, level crossings, superpassages, etc. The Orissa system of canals is situated in a deltaic country, which has a slope somewhat approaching to that of the canals, and, as a necessary consequence, very much fewer heavy works are required than on the Ganges Canal which cross the drainage of the lower Himalayas.

Again, the Henares Canal, in Spain, is stated to have cost, per cubic foot per second, more than twelve times as much as the Mussel Slough Canal in California, but then the works of the former are infinitely superior to the latter. It is very likely that in the end the Henares Canal will be the cheaper of the two, as its annual repairs will cost less, and the works being permanent, there will be no renewals of bridges, aqueducts, etc.

Table 23 is compiled from a table given by Mr. Edward Bates Dorsey, M. Am. Soc. E. C.,* and from other sources of information.

TABLE 23. Giving the cost of canals per acre irrigated, and also the cost per cubic foot per second of discharge.

NAME OF CANAL.	COUNTRY.	COST OF WORKS. Per acre Irrigated.	Per cubic foot per second for water used per year.
Western Jumna	India	$10 88	
Eastern Jumna	India	6 11	
Sutlej or Sirhind	India	26 50	$1765
Ganges (with navigation)	India	36 80	2487
Ganges (without navigation, $\frac{1}{5}$ deducted)	India	28 80	1990
Baree Doab	India	35 00	2330
Sone	India	32 00	2170
Bellary Low Level	India	29 00	1965
Tombaganoor	India	39 00	2600
The Orissa system	India	15 00	1000
Fort Morgan	Colorado		280
Del Norte	Colorado		125
High Level	Colorado	10 83	549
Uncompahgre	Colorado		287
Cajon	California	59 33	1025
Seventy-six	California	6 25	
Santa Clara Valley Irrigation Co	California	9 63	549
Riverside	California	52 75	1507
Mussel Slough	California	7 30	584
King's River, North Side	California	7 18	277
Idaho Mining & Irrigation Co. (estimated)	Idaho	2 16	189
Marseilles	France		4305
Carpentaras	France	35 67	2830
Verdon	France	81 25	15330
Henares	Spain	46 66	7500

*Irrigation in Transactions of the American Society of Civil Engineers. Vol. XVI, 1887.

Article 69. Measurement of Water.—Modules.—Meters.

It is not likely that the greatest duty of water will be reached until it is sold by measure. It will then be to the interest of the user of water to economize it to the fullest extent.

The machines used to measure water in irrigation canals are generally known as modules, or meters. The principal objects to be sought in a module are:—

1. That it should deliver a constant quantity of water with a varying depth or head of water in the supply channel.
2. That it should expend very little head in delivering the constant quantity.
3. That it should be so free from friction as not to be easily deranged, and that sand or silt in the water would not affect its working.
4. That it should be cheap, and so simple in construction that any ordinary mechanic accustomed to that line of work should be able to make or repair one.

It is of great importance that there should be no intricate or concealed machinery, not only from its liability to derangement, but because there is then so much more liability to an alteration in the discharge, without its being noticed by the official in charge. It is also of importance to have, if possible, such a measure as can be easily inspected by those using the water, in order that each man may, if he pleases, satisfy himself that the proper quantity of water is flowing into his channel.

Mr. A. D. Foote, M. Am. Soc. C. E., has invented a water meter which goes very far to satisfy all the above conditions. Professor L. G. Carpenter gives the following description of this water-meter.*

*On the measurement and division of water.

"In Figure 199, *A* is the main ditch with a gate *D*, forcing a portion of the water into box *B*. This has a board on the side towards the main ditch, with its upper edge at such a height as to give the required pressure at the orifice. Then if the water be forced through *B*, the amount in excess of this pressure will spill back into the ditch. If the box *B* is made long enough, and the spill-board be sharp edged, nearly all the excess will spill back into the ditch *C*, thus leaving a constant head at the orifice."

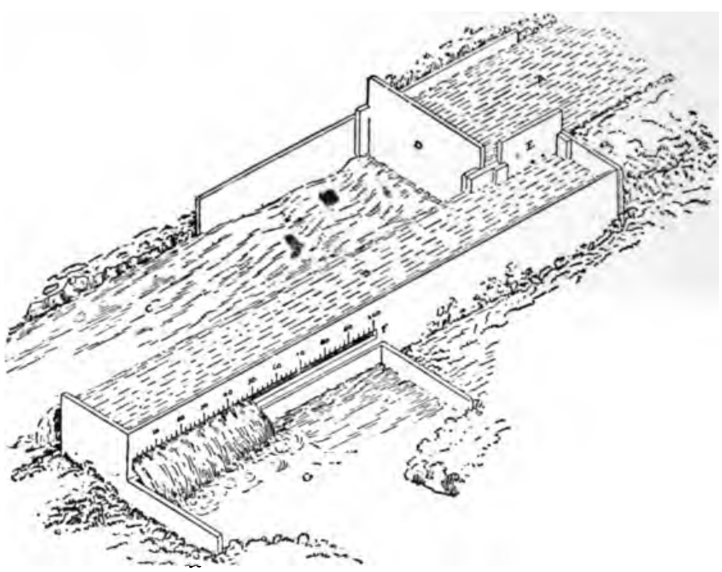


Fig. 199. View of Water Meter, or Module, by A. D. Foote, C. E.

Mr. Foote thus describes this meter:—*

"For months it has done its work in a very satisfactory manner, seldom clogging and never varying in its delivery to an appreciable amount.

"The whole value of the meter depends upon the long

*A Water-Meter for Irrigation in Transactions of the American Society of Civil Engineers, Vol. XVI—1887.

weir, perhaps better described as an excess or returning weir, which returns all excess of water in the box back to the ditch, and thus keeps the pressure at the delivery orifice practically uniform.

"I am well aware that the measurement is not absolutely accurate or uniform; but if it is remembered that the variation in delivery is only as the square root of the variation in head, and that, owing to the long excess weir the variation in head is only a small portion of the variation in the delivery ditch, it will be seen that actual delivery through the orifice is very nearly uniform.

"There need be but an inch or two loss of grade in the ditch, as but very little more water should be stopped than is delivered through the orifice. The gate or other obstruction in the ditch should back the water sufficiently to keep the excess weir clear, and at the same time keep, say, a quarter of an inch of water on its crest, and the surface of the water in the box should then be exactly four inches above the center of the delivering orifice.

"The principle of the long excess weir can be used for delivering water through an open notch or weir, but it is more accurate with a pressure or head, and the greater the head the greater the accuracy, as will readily be seen.

"Any one using the meter will naturally adapt it to their own circumstances and desires. It is cheaply constructed and easily placed in position, costing from four to six dollars; quickly adjusted, as the gates do not have to be precisely set; needs no oversight or supervision (if properly locked as they should be) until a change in volume is desired; will deliver a large or small quantity, which is a great convenience, as the irrigator usually wants a small stream continuously and a large stream on

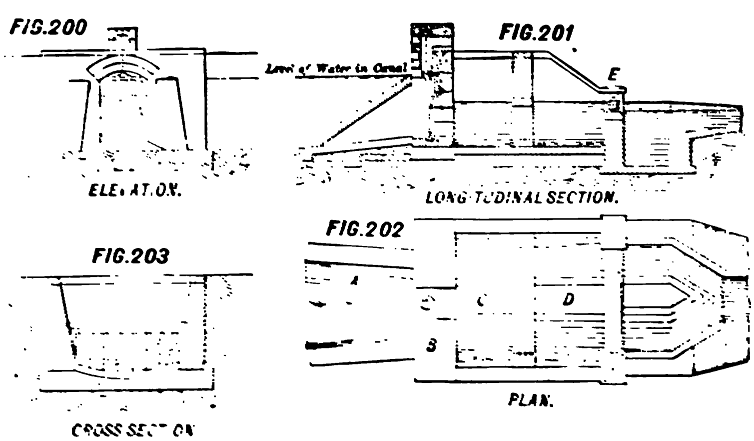
irrigating day; is not likely to clog, as floating leaves and grass pass over the excess weir. Half-sunken leaves may catch in the orifice, but as it is to the farmer's interest to keep that clear, he will probably attend to it.

"To me, however, the greatest merit the method possesses (excepting its accuracy) is that the irrigator himself, with his pocket-rule, can, at any time, demonstrate to his entire satisfaction that he is getting the full amount of water he is paying for."

Whilst Mr. Foote believes that the main ditch need not lose more than a few inches fall, that is from *A* to *C*, Mr. W. H. Graves, C. E., who has introduced the meter on large canals, prefers at least a foot.

The module adopted on the Henares and Esla canals, in Spain,* is illustrated in Figures 200, 201, 202 and 203.

MODULE IN USE ON HENARES CANAL.



"The water is measured by being discharged over a knife-edged iron weir, shown at *E*, Figure 201. The water flows from the main canal into the distributary *A*,

*Irrigation in Spain, by George Higgin, M. Inst. C. E., in Transactions of the Institution of Civil Engineers, Vol. XXVII, 1867–68.

Figure 202, from which place it is admitted into the chamber *C*, by a sluice working in the division wall *B* From *C* the water passes into the second chamber *D*, where the weir is fixed at *E*. The communication between the two chambers, *C* and *D*, is made by narrow slits, and the water arrives at the weir without any perceptible velocity, and perfectly still. The weirs vary from 3.28 feet to 6.56 feet in breadth, according to the quantity of water required to be passed over. On the wall of the outer chamber is fixed a scale, with its zero point at the level of the weir edge, and by means of this scale, any person can satisfy himself that the proper dotation of water is flowing into the distribution channel. By managing the sluice the guard can regulate to a nicety the height of water to be passed over the weir. This module has several good points. The system of measurement is that which possesses the most fixed rules in hydraulics, and gives the most constant results; it is simple, and almost incapable of derangement; it will serve equally well for turbid waters as for clear ones; it can take off the waters with the least possible loss of head—a most important point in countries having a slight surface grade, where the loss of a few feet of headway would prevent the irrigation of many thousand acres. The canal official can see at a glance whether the proper amount of water is passing into the channel, and the irrigators can satisfy themselves on the same point. The only reasonable objection to this module is, that any sudden variation in the head of water in the canal will affect the discharge, which will continue to be greater or less than it ought to be, according to circumstances, until the official comes round again. This is undoubtedly true, * * * but in most well-regulated canals there is never likely to be any serious variation in the head of water in twenty-four hours.

There is, or should be, a man in charge of the head-works, whose special duty it is to see that a constant body of water is admitted into the canal. If the river is flooded, he must close the gates; if it diminishes he must open them. The water taken off from the Henares and Esla canals, for the different water-courses is a fixed quantity, and that passed on to the lower portion is, therefore, likewise variable. The only cause of a sudden change of head would be in the case of a sudden and heavy fall of rain; but to provide against this at every one or two miles, there is a waste weir, or escape, which would immediately carry off the surplus waters; and even if a little more was discharged through the module for a short time, no inconvenience would result from this."

REPORT ON THE PROPOSED WORKS

OF THE

TULARE IRRIGATION DISTRICT, CALIFORNIA,

BY P. J. FLYNN, CIVIL AND HYDRAULIC ENGINEER, MAY, 1890.

To the Honorable, the President and Board of Directors of the Tulare Irrigation District:

GENTLEMEN:—In accordance with your instructions, I have investigated several routes, in order to select the best line, for a canal to convey 500 cubic feet of water per second, or 25,000 miner's inches, under a four inch head, from the Kaweah River to the site of your proposed reservoir. I have also, in this report, according to your instructions, given explanations with reference to objections made to certain parts of the works.

I herewith submit for your consideration plans and profiles and also detailed estimates of the cost of these lines. I also submit tabular statements giving details as to dimensions, grades, etc., of each line. (Only one of these tables referring to Middle Level Canal, No. 1, is given in this pamphlet.)

ESTIMATES.

The estimated cost of each line is as follows:

High Level Canal	$ 744,456
Middle Level Canal, No. 1	659,273
Middle Level Canal, No. 2	664,949
Middle Level Canal, No. 3	669,389
Low Level Canal	695,983

Each estimate includes the cost of head works on the Kaweah River, canal line to reservoir, including tunnels, dam and outlet works at reservoir, canal through the

plains from the reservoir to the district and the compensation to be paid for land for the reservoir and canal lines. To the total cost of the above twenty per cent. has been added, that is, ten per cent. for loss on sale of bonds, and ten per cent. for contingencies. This twenty per cent. is included in the estimates given above.

I recommend the adoption of the line designated Middle Level Canal, No. 1, for the following reasons:

1. It is the cheapest line.

2. With the exception of the High Level Canal there will be less loss of water by percolation than on the other lines.

3. Also with the exception of the High Level Canal, the cost of annual repairs will be less. Briefly stated, the works on this line include head works on the Kaweah River, thence one mile in length of canal to a flume 100 feet long at Horse Creek, thence a canal 2.75 miles long to a tunnel 700 feet long. After this tunnel comes a canal 2,400 feet in length, then follows another tunnel 1,100 feet long and thence 4.59 miles of canal to reservoir. The total length of this canal is 9.15 miles. No water is drawn from the canal between the river and the reservoir. At the reservoir there is a large dam and outlet works, and from the reservoir a canal twenty-five miles long brings the water to and through the Tulare District. The district has an area of about 40,000 acres.

PRICES.

The prices for work are fixed as near the current rate of labor and materials as could be ascertained.

BORINGS AND TRIAL PITS.

In order to make an accurate estimate borings were taken, with a light steel rod, at every hundred feet where the rock was covered with earth. This work was done at slight expense as the ground at the time was

thoroughly saturated with water. A few trial pits were also sunk.

SIDE SLOPES.

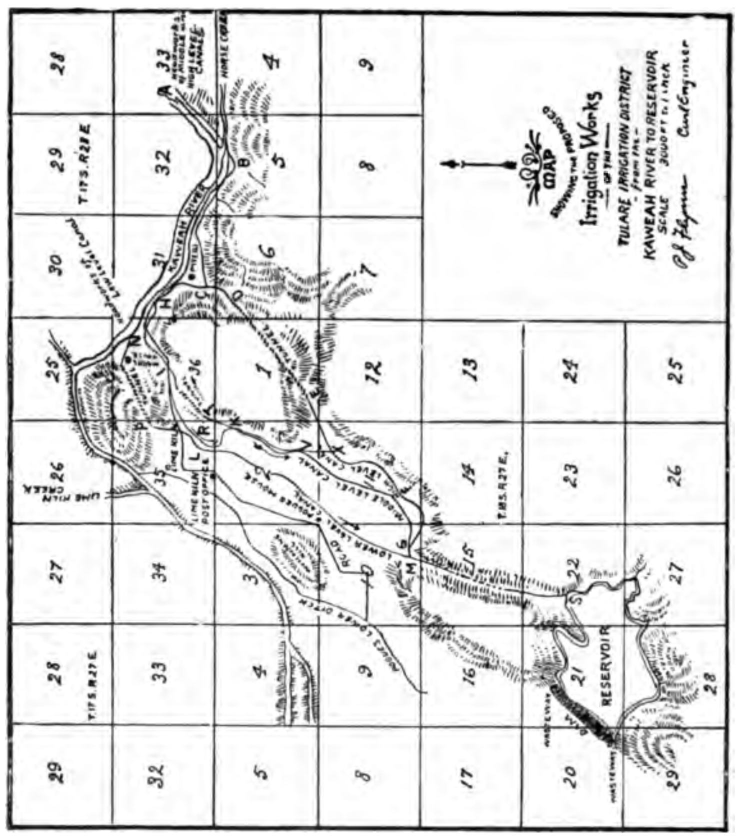
The side slopes in cuttings vary with the nature of the material cut through. In fill the top of the banks is 6 feet in width and $1\frac{1}{2}$ feet above the surface of the water. For one mile from the head works the side slopes, both inside and outside the canal, are 2 horizontal to 1 vertical. With this exception the banks in fill, when not protected by dry rubble, have slope sides of $1\frac{1}{2}$ to 1 on the inside of the canal, and 2 to 1 on the outside. I give a short description of the different lines reported on.

HIGH LEVEL CANAL.

The head of this canal is on the left bank of the Kaweah River in Section 33, T. 17 S., R. 28 E. From this point this canal runs via A, B, C, D, E, X, F, G, M, S, (see map) to reservoir at S. The head of the canal for about 200 feet is through granite, and for the next 5,000 feet to Horse Creek at B, it is through bowlders, gravel and sand. For about 3,000 feet from the Kaweah river this line is in cut and the balance is in fill, about 500 feet in 13 feet fill. This is the largest fill on any of the lines.

Horse Creek is passed by a flume 100 feet long. After this for 500 feet the line runs along a bold rocky bluff, the method of passing which is described under the heading, side-hill work. From this point to D via B, C, D (see map), 7,400 feet in length, the line has frequent sharp curves and runs in steep side-hill ground. The channel is fourteen feet wide at bottom, with a depth of water of seven feet, and with side slopes of $\frac{1}{4}$ horizontal to 1 vertical. This part of the line has been kept as much as possible in five feet cut to prevent loss of water by percolation and breaches. The material cut through

is sandy loam, usually covering, for a few feet in depth, decomposed granite or solid granite. Solid granite shows at the surface at several places, and for about half a mile after leaving Horse Creek Cañon, there are large granite bowlders scattered over the surface of the ground, some of them measuring as much as several cubic yards. In order to avoid the steep side-hill



ground from C to V via C, H, I, T, K, V (see map), the line C, D, E, having a tunnel D, E, 7,500 feet long, was investigated. By this tunnel the line passes through the range of hills that run parallel to, and on the south

side of the Kaweah River. This tunnel is in granite. In cross-section it has a level bed 10 feet 3 inches wide, with vertical sides 7 feet high and a segmental top. Its grade is 1 in 300. From E this canal runs via E, X, F, G, M, S, for 3.7 miles to the reservoir. This part of the line is on fairly level ground, through sandy loam, and no difficulty is met with. This part of the canal has a bed width of thirty-three feet, a depth of water of six feet, the side slopes next to the water 1½ to 1, and the outer slope 2 to 1. The top of the bank is six feet wide and 1½ feet above the surface of the water in the canal. Its grade is 1 in 7,000, and its mean velocity two feet per second. The total length of this line is 7.58 miles.

MIDDLE LEVEL CANAL NO 1.

This canal, which is the line recommended for adoption, is the same as the High Level Canal from the head works on the Kaweah river at A (see map) to C, that is for about 2.5 miles. From C to V, via C, H, I, T, K, V, it runs in a tortuous course, on rough, steep, side-hill ground, through sandy loam, rotten granite and solid granite. Large granite bowlders are scattered over the surface of this route. It will be necessary not only to clear the line of these bowlders, but also to clear the hill side above the canal line of all large bowlders that are likely, during rainy weather, to roll down and fall into the canal. From C to I for 7,800 feet the canal has a bed width of 14 feet, depth of water of 7 feet, side slopes of ¼ to 1, and a grade of 1 in 1000 or 5.28 feet per mile.

At I, this line goes by a tunnel 700 feet long in granite, under the pass near Mr. Marx's house, and it emerges from this tunnel on the south side of the range of hills that run parallel to, and south of the Kaweah River. The lower end of the tunnel is situated at the head of Lime Kiln cañon. This cañon joins at its lower

end with the plain that stretches from the Lime Kiln to the pass M, S (see map), that leads to the reservoir. From I, this line runs along bold, rocky side-hill ground for 2,400 feet to the beginning of a tunnel in granite 1,100 feet long. The method of passing this place is the same as that adopted in passing the rocky bluff near Horse Creek, and is explained under the heading *Side-Hill Work*. From the beginning of the 700 foot tunnel to the lower end of the 1,100 foot tunnel, the line runs through granite. Through this length of 4,200 feet the channel has the same dimensions and grade, that is, in cross-section, bottom level and 9 feet in width, sides vertical and 7 feet high to surface of water. The grade for the tunnels and canal for this length of 4,200 feet is 1 in 200, or 26.4 feet per mile. The velocity in this part of the line is very high, 8.15 feet per second, but the channel is well able to bear this velocity, as it is composed of granite and rubble masonry, the latter having a coat of hard plaster composed of Portland cement and sand. From the lower end of the 1,100 foot tunnel this line falls 22.6 feet in 1,100 feet by 13 vertical drops, and horizontal reaches to K, and the cross-sectional dimensions are the same as the last section, having a bed width of 9 feet. From K the line runs for 4,700 feet to V, along the steep, side-hill ground, through sandy loam and rock. The channel here has a bed width of 14 feet with sides as heretofore described. From V this line runs to X, and thence for 18,400 feet to the reservoir via V, X, F, G, M, S, and from X to reservoir it is the same, in every respect, as the High Level Canal. At V the depth of the canal changes from 7 to 6 feet, and the surface of the water in the channel is assumed to drop one foot near this place. From Horse Creek to V for 4.68 miles the canal has a high velocity sufficient to wash away loams and similar soils.

Where the canal channel, 14 feet in width, passes through these materials the bed and banks have a lining of dry rubble in order to prevent erosion. The superiority of this line over the Middle Level Canals, Nos. 2 and 3, lies in its smaller cross-section and higher grade from K to V, and also in following the line of the High Level Canal from X to S. The depth of cutting through the pass from M to S is less on this line than on lines Nos. 2 and 3. There are two tunnels on this line, one of 700 and the other of 1,100 feet in length. The indications are that these tunnels are in solid granite and will not need timbering or lining. The length of this line is 9.15 miles.

The following table gives the dimensions and grades of the different sections of the Middle Level Canal, No. 1, from the headworks to the reservoir. The velocities are computed by Kutter's formula with $n = .025$. The required discharge is 500 cubic feet per second.

Distance in feet.	Grade 1 in	Slope ft. per mile	Bed width in feet.	Depth of water in feet.	Side slopes.	Area square feet.	Mean velocity in feet per second.	Discharge in cubic feet per second.
5200	2600	2.03	54.	3.5	1 to 2	213.5	2.49	532-(1)
100	200	26.4	16.	4.	Vertical.	64.	8.	513-(2)
400	200	26.4	9.	7.	Vertical.	63.	8.15	513
14200	1000	5.28	14.	7.	¼ to 1	110.25	4.65	513
700	200	26.4	9.	7.	Vertical.	63.	8.15	513-(3)
2400	200	26.4	9.	7.	Vertical.	63.	8.15	513
1100	200	26.4	9.	7.	Vertical.	63.	8.15	513
1100	....	....	9.	7.	Vertical.	63.	8.15	513-(4)
4700	1000	5.28	14.	7.	¼ to 1	110.25	4.65	513
14400	7000	0.754	33.	6.	1½ to 1	252.	2.	504-(5)
4000	....	....	20.	7.	1 to 1	189.	2.55	500-(6)

(1.) This section begins at head works.

(2.) Flume, drop of 0.5 feet.

(3.) Tunnel, drop at tunnel mouth 0.5 feet.

(4.) Level reaches and vertical drops.

(5.) Bed continuous, drop 1 foot in surface water.

(6.) Level reaches and vertical drops.

MIDDLE LEVEL CANAL, NO. 2.

This canal is the same, in every respect, as Middle Level Canal No. 1, from the head works at A to K (see map). At K it drops four feet lower than Canal No. 1, in order to avoid bad ground from K to V. From K to V it is in a slope of 1 in 7,000, whereas Canal No. 1 has in this distance a slope of 1 in 1,000. From V this line runs to the reservoir via V, G, M, S, through moderately level ground. This part of the line is in sandy loam and there is no difficulty in it. There are two tunnels on this line, having a total length of 1,800 feet. The length of this line is nine miles.

MIDDLE LEVEL CANAL NO. 3.

This canal is the same, in every respect, as Middle Level Canal No. 2, from the head works at A to the reservoir at S (see map), with the exception of that part from I to K. From I this canal runs to K via I, R, K, all in open cutting.

This part is 7,000 feet in length. It is very tortuous and runs on steep side-hill ground, through sandy loam and rock. Large granite bowlders are scattered over the surface and embedded in the sandy loam that covers the bed rock. From the lower end of the 700 foot tunnel at I, the line falls forty-three feet in 1,100 feet by fourteen vertical drops and level reaches. Of all the lines this has the shortest length of tunnel, 700 feet. From the last drop below I it runs in a channel of the same dimensions and grade that Middle Level Canal No. 2 has from K to S, and it joins with this channel on the same level at K. It has the greatest length of any of the lines, of difficult, broken, side-hill ground. The length of this line is 9.38 miles.

LOW LEVEL CANAL.

The headworks of this canal are situated on the left bank of the Kaweah River in Section 36, T. 17 S., R. 27 E. The river is here over 500 feet wide and it is divided into two channels. The great body of the water flows in the channel near the left bank, and the river has a decided set towards this bank. There is, however, always danger in such a wide river bed that the main channel, after a heavy flood, might change to the right bank. In such a case it would be very expensive work to excavate a channel from the right branch to the head works of this canal. The head works of Middle Level Canal No. 1 are not exposed to this danger as explained further on. At the side of the head works of the Low Level Canal the left branch of the river is about 150 wide on the surface of the water, and its low water depth about four feet. The site for the head works is in solid granite. A dam in the river, at this place, would be a costly work as, to be effective, it should reach from bank to bank.

From the head works at N this line runs via N, O, P, L, M, S, to the reservoir at S. From the head works at N the line runs across the flat above the left bank of the river to the base of the hill at O. This line is 1,850 feet in length, through sandy loam and hard-pan. This part has a bed width of thirty-one feet, five feet in depth of water, side slopes of ½ to 1 and a grade of 1 in 2,500. From O this line runs through the hill in a tunnel to P for 2,850 feet in limestone. From P the line runs through the plain east of the Wachumna Hill, thus avoiding all side-hill work, to a tunnel under the pass, M, S. The canal through the plain has a bottom width of thirty feet, a depth of seven feet and side slopes of 1 to 1 with a grade of 1 in 8,000.

After passing through the tunnel 3,300 feet in length,

the line runs for 2,000 feet more to the reservoir at S, through sandy loam, hard-pan and granite. There are two tunnels on this line of a total length of 6,350 feet. The tunnels have in cross-section a level bed 10 feet 3 inches in width, vertical sides with a depth of water seven feet and a segmental roof. The grade is 1 in 300. This line is the shortest of the five routes. Its length is 5.59 miles.

TUNNELS.

Under certain conditions a tunnel, when in sound rock, is preferable to an open channel for conveying water. The conditions are that no water is required to be drawn off this part of the line, and that a heavy grade can be given. By sound rock is meant rock not subject to percolation, to any appreciable extent, that will stand the high velocity without injury by erosion, and also that will not require lining for its sides or arching for its roof. When, in addition, a steep grade can be obtained, a high velocity can be given to the water, and the cross-sectional area and consequent expense reduced.

In such a tunnel the loss of water by evaporation and percolation and the expense of maintenance is at a minimum. It has several advantages over the open channel in steep, side-hill ground. Its sides and bed are impervious to water and it is covered from the sunlight. It shortens the line, there is no compensation to be paid for land, and it does not interfere with or cross the drainage of the country on the surface. Should it be required at any future time to increase the carrying capacity of the canal, the discharge of the tunnel can be increased, without, however, increasing its dimensions.

All that will be necessary is to fill all the hollows between the projecting ends of the rocky bed and sides

with good cement concrete, and after this to give a coat of good plaster to the surfaces in contact with the water and make them smooth. Although the section will be diminished, still the velocity and consequent discharge will be doubled.

Let us assume the loss of water in a certain length of open channel at six per cent. of the total flow. If, by adopting a tunnel line, the loss of water is only one per cent., it is evident that it would pay to expend the value of five per cent. of the water on the tunnel line above that on the open channel.

Another argument in favor of the tunnel is, that the amount saved yearly in maintenance capitalized could be expended on the tunnel over that upon the open channel in order to give a fair comparison with the latter. The above are good reasons in favor of the High Level Canal. But, on the other hand, there are two very weighty objections to this route. The principal one is the time the tunnel would take in construction.

Under favorable circumstances, and with granite of medium hardness, this tunnel could be constructed in two years; but, should circumstances turn out unfavorable, and very hard rock as well as water be encountered, the time might be increased to four years and the cost of driving also very much enhanced. The estimated cost of the High Level Canal is $85,000 more than that of the Middle Level Canal, No. 1. If that were the only difference and after taking everything into consideration, then in my opinion the High Level Canal would be the best of the five lines, but on account of the uncertainty as to time and cost, I recommend the next best line, the Middle Level Canal, No. 1.

HEADWORKS OF MIDDLE LEVEL CANAL, NO. 1.

The headworks of this canal are situated on the left bank of the Kaweah River, in Section 36, T. 17 S., R. 27 E. At this site the Kaweah River is well adapted for the headworks of an irrigation canal, in fact, it would be extremely difficult to find in any locality a more favorable location for such a work. It is in a single channel, in a well-defined, permanent, rocky bed, free from sand, silt, gravel and bowlders. The depth of digging at the head is only about eleven feet in rock for about 200 feet in length, and for a mile from the head the greatest depth of digging is only sixteen feet, and this in gravel or bowlders. At low water the greatest width of the river at this place is about 150 feet, and the greatest depth about four feet. At high water its greatest surface width is probably not more than 300 feet. At about 500 feet below this point there is a sudden fall in the rocky bed of the river, and below this fall the channel widens considerably, to 800 feet in some places; and its bed is covered with debris composed of bowlders, gravel, sand and silt. At low stages of the river, in the irrigating season, when water for irrigation is most needed, a large percentage of it is lost by percolation through this porous bed. If, at some future time, in order to economize water and reduce expenses, all canals and ditches on the left bank of the Kaweah River, from Wachumna Hill, to the mouth of Cross Creek, should combine and take out the water from the river in one canal, then this is the proper location for the headworks. With a good permanent dam across the river immediately below the headworks, every cubic foot of water coming down the Kaweah River can be intercepted at this point and diverted into the canal. In seasons of great drought, when every cubic foot of water counts for so much, it is of the utmost importance to be able to

utilize the water that now runs to waste in the porous river bed. The canal can get its supply without a dam in the river, but to be able to intercept all the flow in the low stages of the river, a dam would be necessary. In order to prevent the silting up of the river bed above the dam to its crest, and the choking of the canal head by debris, under-sluices would be required. If the above-mentioned combination of ditch owners should find the building of such a dam necessary, then, by opening these under-sluices in said dam when required, the current will carry away any debris deposited opposite the head gates and keep the latter clear.

For the reasons above given the place selected for the location of the headworks has advantages over every other place that I have seen on the river. These advantages are:—

1. Its elevation above the reservoir is sufficient to give a steep grade to the canal through the bad, rocky ground, and thus diminish its cross-section and expense.

2. The river is in a single, narrow channel, in a permanent bed free from debris.

3. The foundation for the headworks of the most stable and permanent kind, a bed of solid granite.

4. The face line of the head gates can be located on the bank of, and parallel to the direction of the current in the river, and by this means it can be kept clear of the debris.

5. With a dam across the river, and regulating shutters at the head of canal, there will be a command of the water for irrigation, and the water that at low stages of the river is now lost in the bed below can be intercepted and utilized. By closing the regulating shutters at any time the supply can be cut off from the canal and its bed laid dry.

6. On account of the advantages of site above ex-

plained permanent head works can be constructed at moderate expense.

RESERVOIR.

The reservoir has an area, when full, of 657 acres, and it contains 635,340,000 cubic feet of water. Its watershed has an area, including the reservoir, of twenty square miles. It has an earthen dam 56 feet high at the deepest part. Its greatest depth of water is 50 feet, and its average depth 22.2 feet. The dam contains, including puddle and rip-rap, 923,000 cubic yards of material. Its length is 3,800 feet. Its top is 16 feet wide and 6 feet above the level of crest of waste weir that is above the surface of a full reservoir. At the deepest part the dam is 296 feet wide at the base. Its outer slope is 2 horizontal to 1 vertical, and its inner slope facing the water 3 to 1. This slope is to be faced with rip-rap. Under the center of the dam, and for its whole length, a trench is to be sunk to, and into the impervious clayey loam, and afterwards filled with puddle to about two feet above the surface of the ground. The dam will be constructed in thin layers of selected clayey loam well consolidated. An ample waste weir with its crest 6 feet below the top of the dam will be made at each end of the dam, and it will be arranged also so that the outlet can be used as an additional waste channel. The outlet will be through a tunnel in solid rock and through the spur of the hill at the south end of the dam. The outlet will be entirely unconnected with the dam, which will have no pipe or culvert running through it. The tower or chamber connected with the outlet tunnel will be of ample dimensions and of good masonry.

The specifications will enter into more details about materials and mode of construction.

The dimensions given for the dam are those adopted

in the best practice throughout the world. Theory has little to do with the design of an earthen dam. Experience in different parts of the world has shown that with good materials and careful construction a dam of the above dimensions can be made perfectly safe. Statements have been made that there is no necessity for a reservoir, that all that is required is a canal from the Kaweah River to the district, that there has heretofore been ample water in the river for all the requirements of irrigation, and that it, therefore, follows that there will be an ample supply in the future.

In a work entitled "Physical Data and Statistics of California," published by the State Engineering Department of California, there are tables giving the flow of the Kaweah River at Wachumna Hill for six years, from 1878 to 1884 inclusive. The drainage area of the Kaweah River at this place is 619 square miles. From these tables I have compiled the table 1 at the end of this report.

For those more accustomed to compute the flow of water by miner's inches than by cubic feet per second, I give the equivalent of the average flow in that unit of measurement. Fifty of these inches are equivalent to one cubic foot per second. The miner's inch used is that under a mean head of four inches.

From an inspection of these tables it will be evident that the expectation of ample supply in a very dry year, such, for instance, as 1879, is not well founded.

There are over twenty canals and ditches drawing their water from the Kaweah River that will have a prior right to the use of the water, and to which they are legally entitled, before the Tulare Irrigation District can take its supply from that river. I here give the names of some of these canals. They are Wachumna Canal, People's Consolidated, Kaweah Canal, Farmers'

Ditch, Evans' Ditch, Tulare Canal, Packwood, Mill Creek, Outside Creek, Cameron Creek, Lower Cross Creek, Ketchum Ditch, Hayes Ditch, Hambleton Ditch, Meherton Ditch. In addition to the above there are some small ditches not mentioned in this list.

The table shows that the average flow of the river in May, 1879, was only 774 feet. This is the month in which water is most urgently required for irrigation. The only safe rule by which to arrive at the available supply in a year of drought is, to take the least flow of the river when water is most in demand. The canals and ditches mentioned above are entitled to more than 774 cubic feet per second. It is very likely that all the canals have never drawn the full supply to which they are entitled at the same time during the period of least supply. The time, however, is sure to come when they will do so. As the country is thickly settled the demand for water will increase until every available cubic foot that can be drawn from the river will be utilized on the land.

Under these circumstances, in a very dry year it is evident that there will not be sufficient water to save the crops that are depending for their supply on the canal alone. In such a deplorable state of affairs the loss to the district in a year of great drought would be more than the total cost of the reservoir. The reservoir is intended to insure a supply during the period of the low stage of the river, and to prevent a water famine on the irrigable lands of the Tulare Irrigation District.

The canal alone no doubt will bring a supply during years of average or more than average rainfall, but it is sure to fail when most required in seasons of great drought, for the sufficient reason that there will be no water supply.

During this year there is an abundance of water avail-

able, but it is well to remember that we are after having a most unusual wet winter, and it is of still more importance to remember that extraordinary seasons of drought happen periodically, and that in only one such season the use of the storage water from the reservoir will more than repay the expenditure incurred on the dam. Without a reservoir in such a year the canal will be a dry channel unable to supply the perishing crops with water.

In the months between irrigating seasons, when there is not such a large quantity drawn from the river for irrigation, the water that now runs to waste, during that period, can be taken to fill the reservoir, and there will thus be a storage reserve to be used only when it is urgently required in April and May. In the meantime, after the reservoir is full, any water that may be drawn from the river can be allowed to flow down to the district and be used for irrigation. For instance: Let the reservoir be filled at the end of the irrigating season, when there is always an abundant supply of water in the river from the melting snow. Now, from this period until the following irrigating season in April, the supply obtained from the river flows to and out of the reservoir, keeping it full. In case, however, that the supply from the river should at any time in a dry year fail, there will still be a full reservoir stored for use.

In average years, however, the reservoir can be filled several times from freshets and melting snow, and by this means at the periods of irrigation there will be a larger supply available at certain intervals than could be obtained from the river by the canal alone.

The above facts prove that to have, in all seasons, an effective system of irrigation works for this district, a storage reservoir is essential.

Statements have been made that, after completion, a

full reservoir would not be capable of irrigating one section, that is, 640 acres of land. I now proceed to prove that these statements very much exaggerate the probable loss from evaporation and percolation. The quantity of water required to irrigate land varies very much. The number of acres that a cubic foot per second, or fifty miner's inches will irrigate is known as

THE DUTY OF WATER.

This varies from 50 in wheat lands, in some parts of America, to 1,600 in fruit land in Southern California. When this high duty is reached the water is conducted in pipes, and it is used with economy.

In Elche, in Spain, where water is very scarce, a cubic foot per second irrigates 1,000 acres of land.

General Scott Moncrieff, R. E., gives the duty that can be got out of one cubic foot of water per second in Northern India, at 250 acres, and he states that there is frequently fifty days between each irrigation.

J. S. Beresford, C. E., states that five inches in depth is a safe allowance for one watering in Northern India. I have heard an experienced irrigator in this district, Tulare, state that he gave over six feet in depth, at one watering, to a piece of land having a sandy soil. He had an unlimited supply of water and the quantity used he measured from the supply channel.

Prof. George Davidson in his Report on Irrigation, states that:—

"The amount of water required for a crop of wheat, barley, maize, etc., is almost identical with the amount deduced from observations in the great valley of California, where a rainfall of 10⅓ inches, fairly distributed, will insure a crop."

"The capacity of a canal may, therefore, be fairly estimated by assuming that 12 inches of water over the

surface of the irrigable land will, if properly applied, be amply sufficient for the maturing of one grain crop; and hence, knowing the capacity of a canal, we can determine the area its water will irrigate."

In one of his lectures before the Academy of Sciences at San Francisco, Prof. Davidson says on the same subject:—

"In estimating the total acres that can be irrigated from a given supply, allowance must be made for the amount lying fallow, woodland, marsh, roads, streams, towns, etc. In India, the average under cultivation each season is only one-third of any given area; in this country we might safely estimate it at two-thirds of any irrigation district."

Experienced irrigators state that in this district, as a rule, one watering some time in May will save the crops, vines and fruit trees, and that fruit trees and vines can, with careful cultivation, tide over one dry season, with less than an average depth of six inches over the land. From the instances given it will be seen that there is a wide diversity in the quantity of water used per acre to irrigate land.

The reservoir, when full to the level or waste weir, will contain 635,340,000 cubic feet, equivalent to 4,752,660,870 U. S. standard gallons of water. If we reduce this quantity by thirty-six per cent. for evaporation and percolation, up to the point of delivery to the irrigators, we have left for purposes of irrigation 406,617,600 cubic feet. This is the quantity that, after the loss by evaporation and seepage, would be given to the irrigators for use on their land in a very dry year in April or May. This quantity is sufficient to cover 11,200 acres to a depth of ten inches or 18,600 acres to a depth of six inches.

It is the opinion of irrigators well informed on the

requirements of this district, that this quantity of water, used with economy, would be sufficient to save the crops, fruit trees and vines, and tide over a very dry year in this district.

Doubtless, during the first few years after the opening of the canal the loss of water will be more than thirty-six per cent., but as explained under the heading *Evaporation and Percolation*, the loss from seepage will decrease with the age of the canal and also as the sub-soil gets saturated with water. The Fresno District is a notable instance of the saturation of sub-soil. A small percentage of the quantity of water used at first to irrigate a certain area is now sufficient to insure a crop.

I am informed that the distribution channels that I constructed in 1877, to irrigate the twenty acre lots of the Central California Colony at Fresno, have since been leveled and filled up, as the sub-soil is so saturated with water that very little flooding is now required. There is a deeper porous sub-soil in this district and, therefore, it is not likely that its saturation will be to the same extent as that of Fresno, but it will probably be sufficient to diminish the quantity of water now required to irrigate a certain area in this district. I am informed that already there is a sensible rise in the sub-soil water, in and around Tulare, which is attributed to the seepage from the irrigation channels in the district.

The storage capacity of one full reservoir, at a time when there is no additional supply flowing into it from the Kaweah River, would supply a canal having a discharge of 500 cubic feet per second or 25,000 miner's inches for fifteen days, and, when there is no outflow from the reservoir, it would take an equal length of time for the supply canal from the river to fill it.

In the period of greatest demand for irrigation, in years of ordinary rainfall, there will be a supply from

the river, flowing into the reservoir to add to its greatest storage reserve. This supply from the river will be a material addition to the irrigating capacity of the reservoir.

As an instance let us assume that during the period of irrigation 500 cubic feet per second are drawn from a full reservoir, while it is, at the same time, receiving 200 feet per second in excess of all losses, including evaporation and percolation. In this instance the reservoir and canal combined will give a supply for irrigation of 500 cubic feet per second for twenty-four and one-half days and will, during this time, cover 24,297 acres to a depth of one foot.

Without the reservoir the 200 cubic feet per second supplied by the canal would cover two-fifths of that area, equal to 9,719 acres.

Without the additional 200 cubic feet per second by the canal, the reservoir alone would give a supply of 500 feet per second for fifteen days, and would cover 14,585 acres to a depth of one foot. I append a table showing the great increase of the irrigable capacity of the reservoir supplemented by a supply from the river.

The first column of the table gives the number of cubic feet per second supplied by the canal from the river to the reservoir.

The second column gives the number of days supply for irrigation at the rate of 500 cubic feet per second, that the full reservoir of 635,340,000 cubic feet can give when supplemented by the quantity in column one.

The third column gives the number of acres that can be covered to a depth of one foot by 500 cubic feet per second, in the number of days given in second column.

The fourth column gives the number of acres that the canal supply in the first column, but without the reser-

voir, can cover to a depth of one foot in the number of days given in the second column.

Canal from Kaweah River, cubic feet per second.	Reservoir and Canal gives a supply of 500 cubic feet per second for days	Reservoir and Canal cover to a depth of one foot acres	Canal alone without reservoir cover to a depth of one foot acres
...	15.	14,585	....
50	16.3	16,202	1,616
100	18.4	18,235	3,650
150	21.	20,833	6,248
200	24.5	24,297	9,719
250	29.4	29,157	14,570
275	30.	29,759	15,174
300	36.8	36,483	21,897
350	49.	48,595	34,016
400	73.5	72,899	58,314
450	147.	145,792	131,206

An inspection of this table will show the necessity for a reservoir in a dry year. With a supply of 100 cubic feet per second, 5,000 miner's inches, the canal alone, in 18.4 days will cover 3,650 acres to a depth of one foot, whilst the reservoir, plus this supply, will irrigate 18,-248 acres to the same depth in the same time. In the former case there would be blighted crops over a large area, and in the latter, on the contrary, there would be sufficient water, if used economically, to save the crops throughout the district.

LOSS FROM EVAPORATION AND SEEPAGE.

There is a popular belief that the loss of water from the surfaces of rivers, canals and reservoirs is much greater than is actually the case.

The records of evaporation at Kingsburg bridge, Tulare county, published by the State Engineering department of California, are given in the tables at the end of this report. From Table 12 it will be seen that the mean annual evaporation at Kingsburg bridge for the four years from 1881 to 1885 is 3.85 feet in depth,

when the pan is in the river, which is equal to an average depth of one-eighth of an inch per day for a whole year. For the same period the evaporation, when the pan was in air, was 4.96 feet in depth, that is, equal to a mean daily depth of evaporation throughout the year, of less than three-sixteenths of an inch per day.

The greatest evaporation is in the month of August, when it is more than one-sixth of the evaporation for the whole year. The average for this month is one-third of an inch per day.

During the months when the largest quantity of water is used for irrigation in this district, the table shows that the mean evaporation is:—

For March one-twelfth of an inch per day.

For April one-twelfth of an inch per day.

For May one-fifth of an inch per day.

To some people these depths of evaporation may appear very small. Let us, therefore, examine the result of observations in other countries:—

Colonel Baird Smith, in his work on Italian Irrigation, states that in the north of Italy and center of France, the daily evaporation varies from one-twelfth to one-ninth of an inch per day; while in the south, and under the influence of hot winds, it increases to between one-sixth and one-fifth of an inch per day.

In July, 1867, the evaporation in Madrid, according to the returns of the Royal Observatory, was 13½ inches in depth, or less than half an inch per day; and in May of the same year it was only one-quarter of an inch per day. July was the hottest month in 1867, and it was estimated that during this month the total evaporation of the Henares Canal, carrying 105 cubic feet per second, or 5,250 miner's inches, amounted to only three-fourths of one per cent. of the total flow.

W. W. Culcheth, C. E., states as the result of his in-

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vestigation on the Ganges Canal, in Northern India, that for evaporation, one-quarter of an inch per day over the wetted surface may be taken as the average loss from a canal.

Dr. Murray Thompson's experiments in the hot season in Northern India, with a decidedly hot wind blowing, gave an average result of half an inch in depth evaporated in twenty-four hours.

M. Lemairesse's observations at Pondicherry, in French India, give a daily evaporation of from three-tenths to half an inch in depth per day.

Trautwine made observations in the Tropics and he found the evaporation from ponds of pure water to be at the rate of one-eighth of an inch per day, but he observes that the air in that region is highly charged with moisture.

The above quoted observations, although they do not prove the accuracy of the Kingsburg experiments, still they give results, in warm climates, so close to each other that, for all practical purposes, the latter experiments may be accepted as correct.

Let us now investigate the loss of water from the reservoir by evaporation:—

Let us assume that the reservoir is full on the 31st of July, and that it receives no water from the river from this time until it is drawn upon for water for irrigation on April 1st, of the following year. Allow twenty days for the reservoir to become empty and the surface exposed to evaporation during this time is equal to the surface at full supply for half the time, or ten days. From Table 12 we find the average evaporation for four years to be as follows:—

August	0.861	feet in depth
September	0.615	" "
October	0.289	" "
November	0.174	" "
December	0.104	" "
January	0.081	" "
February	0.091	" "
March, one-third of	0.075	" "
	2.290	

This shows a total depth evaporated during this time equal to 2.29 feet, but the average depth of the reservoir at full supply level is 22.2 feet, and therefore the evaporation is, in round numbers, 10 per cent. of the full reservoir.

To this will have to be added evaporation of twenty days in April in the main canal and small ditches, during the time that the reservoir is being emptied.

If we take the length of the main canal at 40 miles and the width of water surface at 66 feet, we have an area of 320 acres, and if we allow the same area for the smaller ditches, we have 640 acres for twenty days in April, or in round numbers, the same area as the reservoir, 657 acres. The evaporation, Table 12, is given as 0.286 feet in depth for the whole month of April. As the water from the reservoir will pass over the heated, dry bed of the canal, let us allow the evaporation for the twenty days in April to be as much as that of the whole month, or 0.286 feet in depth. This depth on 657 acres is equal to 1.3 per cent. of the mean depth of the reservoir. This shows that the evaporation from reservoir and channels below reservoir is less in volume than 12 per cent. of the full reservoir.

The observations made last year at the Merced reservoir are in support of these deductions, the evaporation having been found less than that given in Table 12.

There is usually more loss of water from seepage in earthen channels than from evaporation.

In every new canal, through sandy loam, the loss by absorption at first is very serious. Gradually the ground gets saturated, and at the same time the interstices of the porous material of the bed and banks get filled up with particles of clay, which diminish the percolation. The bed of a canal acts as an *elongated filter*. It is well known that the sand of a water-works filter-bed, if it is not periodically washed, or replaced with clean sand, the interstices between its particles get filled with silt and the filter ceases to act, or acts so slowly as to be practically useless. The same thing takes place in a canal, but at a slower rate than in a filter-bed. There is less deposit in a canal, as the greater part of the finer particles of silt do not subside until the water reaches the land to be irrigated.

At first, after the completion of the canal, probably not more than 25 per cent. of the irrigable land of the district will require water. Gradually, as time goes on, small fruit farms will increase in number, and with them the area of land requiring water. At the same time the percentage of loss by percolation will decrease, and a larger quantity of water will be available than at the first opening of the canal.

The loss by percolation will be most serions in the sandy reaches of the canal. These sections can be taken in hand and puddled, one at a time, during the annual repairs, and the puddling thus spread over several years and charged to the working expenses. The puddling can be done at a time when the canal is not used for irrigation purposes.

Ribera estimated the total loss from evaporation and percolation in the Isabella Canal, in Spain, a masonry-lined channel, at two per cent.

In the Ganges Canal, in India, the largest irrigation canal in the world, with a discharge of 5,000 cubic feet

per second, or 250,000 miner's inches, the loss in 1873-74, from all causes, including evaporation, seepage and waste, was 69 per cent. The length of main and branch canals of all sizes was, however, at this time, over 4,000 miles long. The length of main canal alone was 648 miles. It is admitted that water was very wastefully used, and this, together with the great length of the channels, accounts for the extraordinary loss.

P. O'Meara, C. E., in writing on the results of irrigation in this country, attributes the principal loss of water to evaporation, and he states that:—

"The question of evaporation was so important that it was doubtful if any loss of irrigating power occurred in Colorado, other than that which was due to it."

Walter H. Graves, C. E., in a paper read before the Society of Engineers, in Denver, Colorado, in 1886, states:—

"The factor of seepage is a variable one, depending mostly upon the nature of the soil, and gradually grows less through a long term of years. Evaporation is very nearly a constant quantity. * * * In calculating the loss from these sources in the older canals, about twelve per cent. should be deducted from the carrying capacity. Observation and experiment by the writer in various parts of Colorado, tend to show that evaporation ranges from .088 to .16 of an inch per day, during the irrigating season."

From what has been written, it will be seen that the loss from evaporation and seepage combined varies from twelve per cent. in Colorado to sixty-nine per cent. in India. As a fair average, therefore, thirty-six per cent. is allowed for the loss of water from these causes, in computing the capacity for irrigation of the reservoir and canal of the Tulare Irrigation District.

EARTHEN DAMS.

Several objections, not founded on facts, have been urged against the reservoir, and it has been stated that an earthen dam cannot be built to impound water at a depth of fifty feet. This is a mistake. Facts prove the contrary. There is no good reason to doubt that what has been well done before in thousands of cases, in putting a large quantity of good clayey loam together to retain water, can be done again.

There are thousands of earthen dams in different parts of the world, impounding water to a depth of 50 feet or more, that are in use to-day, and that possess as much stability as the modern brick houses in which are living millions of people. Properly constructed, on a good foundation, and with a waste weir large enough to carry off the greatest rainfall, an earthen dam can be constructed to have as much stability, and as long a life, as any iron railroad bridge in the country.

An ample waste weir is the safety-valve of a reservoir.

If, by any means, the waste weir is contracted so as to diminish its discharging capacity below its requirement for the maximum rainfall, then the dam is in danger. There would be as much sense in bolting down the safety valve of a steam engine, as in obstructing a waste weir of a reservoir, and still we read that the latter has been done and caused frightful loss of life.

The top of the dam must also be kept to the level of its original height above the crest of the waste weir. Allowing the top of the dam to settle below its intended height is just as bad as raising the waste weir an equal distance. In the construction of the dam provision must be made for settlement by adding a certain percentage to its height.

It is as essential to keep a dam, its waste weir and outlet in repair, as it is to keep a house or bridge in the

same condition. Some people assert that if a dam is built it should be of masonry, as, in their opinion, this is the only material that can safely resist the pressure and erosion of the water.

A masonry or concrete dam requires to be founded on the solid rock. Clay or impervious clayey loam is not a suitable foundation for it. This is the reason why, in so many instances, the underground work on a masonry dam has cost more than that above ground, as it has to be taken down to bed rock. A masonry dam founded on clay, or other compressible material, is likely to settle and crack, and thereby cause serious trouble and expense, or its total destruction. On the other hand, while an earthen dam can, with safety, be founded on solid rock, still, its best foundation is in good clay, clayey loam, hard-pan or other similar material impervious to water. The reservoir dam proposed has for the greater part of its length a good foundation, at little depth, in clayey loam or hard-pan, and for this reason an earthen dam has been selected as being the most suitable for the location.

During the last few years railroad bridges have broken down under passenger trains, causing fearful loss of life, and Buddenseick buildings have tumbled down either during erection or soon after completion. These accidents have not prevented people from traveling by rail or living in brick houses.

Experience has proved that an earthen dam can be constructed so as to be as safe and stable as any bridge or building in the world. In this State, the Merced dam and the dams of the Spring Valley Water Company of San Francisco, are examples of safe construction. Some of them are in use for over twenty years.

India can show thousands of dams that have been in use for over a century, and that are perfectly safe now.

In the Presidency of Madras, the official records show that there are over 43,000 reservoirs in use at the present time. In an official return issued by the Irrigation Department of Bombay, on the 1st of September, 1877, there is a list of seventeen dams, either completed or in progress of construction, the lowest of which is 41 feet and the highest 101 feet high, and this is the work of only a few years. This shows that the Indian engineers have, from long experience, the fullest confidence in the stability of their earthen dams. Is it to be credited that the progressive American of the present time is not able to construct an earthen dam as well as the natives of India of the last century?

As pertinent to this subject an extract is given from a paper by the writer, published in the Transactions of the Technical Society of the Pacific Coast for June, 1885, on the

SHRINKAGE OF EARTHWORK.

"Embankments in India are often constructed by basket work, the material being carried in saucer shaped wicker baskets, containing less than a cubic foot. In the construction of embankments to retain water, this basket work is done in thin layers of less than nine inches in depth, the earth being roughly leveled up as it is deposited from the baskets, and then well punned with wooden or cast-iron rammers, weighing about twelve pounds. In addition, the constant tramping of the men, women and children employed in carrying the baskets, so consolidates the bank as to make it impervious to water. The layers of earth are sometimes watered. Embankments constructed in this manner shrink or settle very little after they are finished. They are, in fact, an approach to puddle work, though not nearly so expensive. The writer has constructed many

embankments with a grading machine, tipping from wagons from grade, wheel-barrows, hand-cars, carts, scrapers and punned basket work, and of all these he believes that punned basket work settles the least, and is the best suited for hydraulic work, and the next best work to it for a similar purpose is that done by scrapers.

"Thousands of embankments, and some of them counted among the largest and oldest dams in the world, have been constructed in India, by basket work, without any puddle wall or puddle lining; and some of them, that have been looked after and kept in repair, are as good, if not better, at the present day than when they were originally constructed, hundreds of years since. This kind of work is done much cheaper there than earthwork in this country.

"The writer has constructed embankments in the Punjab, the lead being from 100 to 200 feet, for three rupees per thousand cubic feet, that is, at the rate of four cents per cubic yard."

The numerous reservoirs for the water supply of cities all over the United States are proof that earthen dams in large numbers, and of a greater height than 50 feet, have been constructed during the last forty years in America.

These dams are as safe as the ordinary railroad structures, and many of them are located in the midst of a dense population.

The failure of earthen dams in the United States is mainly due to the cupidity of companies or corrupt contractors. Another cause of failure is the too common belief that any ordinary laborer, that any man who has used a scraper on a county road, is fitted to superintend the construction of an earthen reservoir dam. Materials that would in some instance be suitable for a county road or a railroad embankment, would be likely to cause

destruction to a reservoir dam, and even with proper materials more care has to be taken in constructing the latter than the former, and also a different method of raising the embankment has to be adopted.

It is safe to assert that over fifty per cent. of all the dams in the world, constructed as part of the works for the water supply of cities are built of earth.

Long experience has shown the dimensions required for dams. With these dimensions, good material properly put together, a tower and outlet pipe through a tunnel in solid rock, the face of the dam covered with rip-rap, and an ample waste weir, an earthen dam can be made as safe as any structure on the best constructed railroads.

Some anxiety has been expressed about danger to the dam from gophers, but it appears that they do no sensible damage to the Merced or Spring Valley dams of this State already referred to.

CANAL ON STEEP SIDE HILL GROUND.

In the description of the different lines already given, frequent mention is made of steep side-hill work. Following the lines of the canal on the map from Horse Creek by the distinguishing letters, B, C, H, I, T, K, V, and also the loop, I, R, K, almost all the work for this distance is on steep side-hill ground, the slope in some instances being as high as twenty-six degrees, that is, a slope of two horizontal to one vertical. The material is sandy loam, hard-pan, disintegrated rock and solid granite. The depth of the rock from the surface varies considerably. Sandy loam is usually a surface covering of the other materials, and varies in depth from a few inches to six feet and more. It is much more difficult to carry a canal discharging 500 cubic feet of water per second, or 25,000 miner's inches, along such ground than it is to carry a railroad or county road.

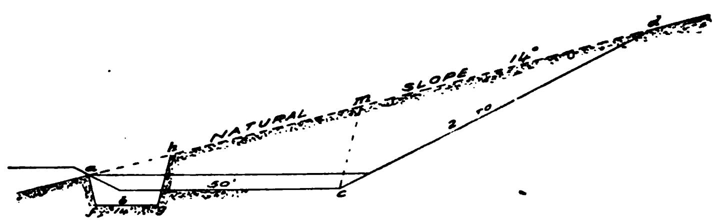
Hydraulic miners, who have had to construct ditches and keep them in repair, know the great difficulty and expense of keeping a ditch to convey twenty-five cubic feet of water, or 1,250 miner's inches, in repair. How much more difficult, then, must it be, to convey twenty times that quantity in one ditch, that is 500 cubic feet of water per second. Other things being equal, the less the cross-sectional area of the channel, the less will be its cost, and the less the annual expense for repairs, when the velocity is kept within the limiting resistance of the materials of which the channel is composed, that is, when it is not so great as to abrade the bed and banks.

These considerations led to the adoption of the cross-section having a bottom width equal to twice the depth, for the steep, side-hill work, with the exception of the crossing of the bluff at Horse Creek, and that part of the line between the 700 and 1,100 feet tunnels. The dimensions are, bed width 14 feet, depth of water 7 feet, side slopes ¼ to 1. With a grade of 1 in 1,000, that is, 5 feet 3 inches per mile, the velocity in this channel, according to Kutter's formula with $n = .025$, is 4.65 feet per second, and the discharge 500 cubic feet per second. The levels through the hills admit of the grade given without adding materially to the length of the tunnels, and the material cut through is, on the whole, suitable for a high velocity. When the material cut through is sandy loam, or other materials that the high velocity of 4.65 feet per second in this canal would wash away, protection is afforded the banks by a facing of dry rubble masonry, and the bed will be protected with stone paving. Rock is in abundance all along the hillsides for this work.

It has been stated that this channel will not discharge 500 cubic feet per second, and that a more suitable one would be a section with a bed of 50 feet, a depth of

water 3½ feet, with side slopes of 2 to 1. The latter section, with a slope of two feet per mile will, according to Kutter's formula with $n = .025$, give a velocity of 2.5 feet per second and a discharge of 500 cubic feet per second. The small section will discharge just as much as this large one, and its cost will be much less. The principal objection to the large section is its expense.

Fig. 205.



Cut afgh - 151 sq ft.
abcd - 1218 " "
abcm - 634 " "

Figure 205 is a diagram drawn to a scale of thirty feet to the inch, showing the two channels on side-hill ground. The slope of the ground is fourteen degrees. This is about an average slope on the bad ground. The cross-section *a, f, g, h*, adopted for the steep side-hill ground in this report has an area in round numbers, of 151 square feet, and the cross-section of *a, b, c, d*, with a bed 50 feet wide, and side slopes of 2 to 1 has an area of 1,218 square feet, that is, in the latter, about eight times as much material will have to be moved as in the former section. If, again, the slope *c, m*, be made ¼ to 1, then the area *a, b, c, m*, is equal to 634 square feet, that is more than four times as much as the section adopted.

The large section is not, under any circumstances, the right one for steep side-hill ground, although it is, in some cases, suitable for a canal in the plains. An in-

spection of the cross-sections in Figure 205, will make this very evident.

In Figure 205, the cutting is made of such a depth that the water is all in soil, that is, that the depth of cutting is made equal to the depth of water in the channel. If, however, the canal, instead of being in cutting is in embankment equal to or less than 3½ feet, and the surface of the water in the small channel, be at the same level as in the larger one, the advantage is still in favor of the smaller section. Where it can be done with advantage the intention is to keep the adopted section *a*, *f*, *g*, *h*, in about five feet depth of cutting, at the point, *f*, that is, that the vertical depth of the bed at *f* will be five feet below the surface of the ground at *a*. As the depth of water in this section is 7 feet there will be 2 feet in depth of water in embankment. The top of the bank will be 6 feet wide and will be 1½ feet above the water, and its outer slope in earth 2 to 1, that is, for every two feet horizontal there will be one foot vertical.

If necessary, the cross-section will be varied to suit the ground, keeping the depth of water seven feet in all cases in side-hill ground.

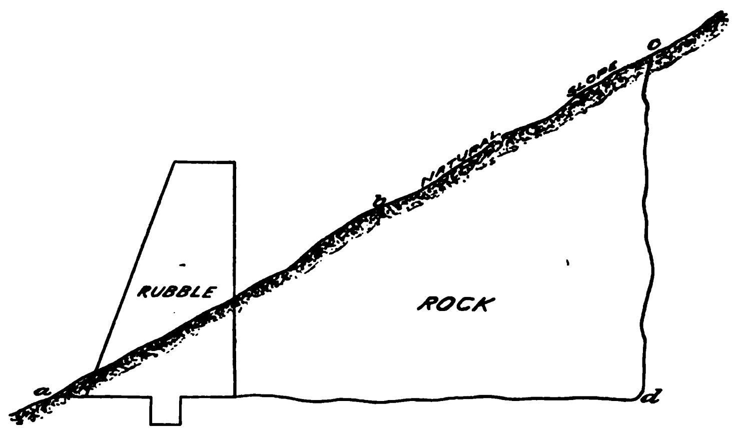
As a rule, the best but most expensive plan, for a canal in loamy soil in steep side-hill, is to put the section in cut equal to the full depth of the water.

In passing the bold rocky point at Horse Creek, and also in that portion of the line between the tunnels I and T, the rock will be taken out in the shape of a right-angled triangle, as shown in cross-section at *a*, *d*, *c*, Figure 206. Then a wall of uncoursed rubble masonry in lime mortar will be built on the lower side. The inner side of this wall will have a coat of plaster composed of Portland cement and sand.

As an additional precaution to prevent percolation, a groove will be cut in the rock under the wall, which

groove will be filled with concrete and this concrete will be joined with and form part of the rubble wall. The cross-section of the channel inside the rubble wall will be nine feet on a level bed, with vertical sides. The wall will be eight feet high, two feet wide on top and

Fig. 206.



five feet on the bottom, with the side next the water vertical and the outside battered. The grade of this channel will be 1 in 200 or 26.4 per mile. The cross-section and grade of this channel, from its bed to the surface of the water, will be the same as that of the tunnels at each end of it.

This section of the line from the upper end of the 700 foot tunnel to the lower end of the 1,100 foot tunnel will be the best part of the line. There will be less loss of water by evaporation and percolation, less expense in annual repairs, and less danger of breaches than in any other part of the line through the hills of an equal length.

It has before been mentioned that the discharge through the tunnels can be doubled by giving the bottom and sides a smooth plastered surface. The same

thing can be done in the channel 2,400 feet in length, between the two tunnels which has the same sectional area as the latter. This is a fact well known to hydraulic engineers, that the new and improved formulæ give an increased discharge in proportion to the smoothness of the material over which the water flows. This fact was not taken into account in the old formulæ, which are now known not to give the true discharge under all conditions of channel.

Materials for building the rubble will cost very little. After the excavation there will be sufficient rock for the work at hand; the bed of the Kaweah River, a short distance away, will supply the sand required; lime is burned within half a mile of the work, and water is in abundance in Mr. Pogue's ditch. In the remainder of side-hill ground from Horse Creek to V, a similar section, 16 feet wide on bottom, can, no doubt, be adopted in several places, but this can be ascertained only after the surface covering of sandy loam is removed. This part of the line has a grade of 1 in 1000, or 5.28 feet per mile.

A level bed 16 feet in width, with vertical sides, holding seven feet in depth of water with this grade, will, according to Kutter's formula, with $n = .025$, give a velocity of 4.53 feet per second, and a discharge of 507 cubic feet per second.

RAINFALL.

An inspection of the rainfall of Tulare given in tables* will show that, during the ten years from 1874 to 1884, three years had total depth of rainfall for each year of less than four inches, three years of less than seven inches, three years of less than ten inches, and one year

* Tables not given in this Report.

of 11.65 inches, which was the maximum during this period.

This plainly shows the necessity for irrigation in this district and nothing further will be said on this subject.

A rainfall of 10 to 12 inches properly distributed will mature a crop in this district.

The catchment basin of the reservoir, including the area of the latter, is 20 square miles. The reservoir is a little more than one square mile in area.

In order to show the height which a large rainfall would raise the reservoir, let us assume that the reservoir is full to the level of waste weir, and that no water can flow out of it through the waste weir or otherwise. In this state of affairs let the canal flow 500 cubic feet per second for one hour, whilst at the same time an extraordinary rainfall of three inches per hour takes place, of which 50 per cent. reaches the reservoir. At the end of one hour the reservoir would have risen 2 feet 7 inches, leaving the top of the dam 3 feet 5 inches above the surface of the water.

This is a state of the reservoir not possible under any circumstances. The waste weirs will be always open and discharging to the capacity of the depth of water on them. A very heavy rainfall usually lasts but a short time, and the large capacity of the reservoir prevents a dangerous elevation of its surface during heavy rains.

The total length of the waste weirs at each end of the dam must be made of sufficient capacity to carry off the flood water (computed by Dicken's formula), in addition to 500 cubic feet per second from the Kaweah River. A sufficient length of waste weir will be given so that the depth of water on its crest will not be greater than three feet. This would leave the top of the dam three feet above the surface of flood water.

PREVENTION OF WASTE OF WATER.

Two methods have been adopted to prevent waste of water. One by measurement and the second by making the irrigators raise all the water they use from the supply canals. There are two methods used in India, in supplying water, known as *flush* and *lift*. In flush irrigation the water flows by gravitation on to the land to be irrigated. In lift irrigation the water reaches the land at such a low level that it cannot flow over the surface of the land to be irrigated. This requires power of some kind, usually manual labor, to raise the water sufficiently to enable it to flow over the land. So great was the loss from waste in India some years since, that it was seriously proposed to supply all the water at such a level, that it should be lifted some height, however small, before it could be used. It would then be to the interest of the irrigators to prevent waste. In this country the best method to prevent waste is by

MEASUREMENT OF WATER.

A meter for measuring water for irrigation purposes must be cheap and simple in construction and must cause little loss of head. No machine has yet been invented that fulfills all these conditions. The great difficulty is the fluctuation of the level of the water in the main canal.

It is believed, however, that a machine can be devised to fulfill these conditions that will give a close approximation to the quantity of water used. When the same method of measurement is used toward all the irrigators they will be treated on perfect equality and no one will have good reason to complain of injustice more than another.

DRAINAGE.

As a rule the drainage of irrigated land will take care of itself, if the natural drainage channels are left free and unobstructed. These channels should not be used as drainage channels and also for purposes of irrigation. Nature, the best engineer, located them to convey water from the land. They cannot, with advantage, be used for irrigation and drainage, and when so used the worst results invariably follow.

If the sub-soil and surface water cannot escape freely by the natural channels, super-saturation follows and the ground becomes water-logged.

We have not to go far to see the evil effect of too much irrigation with defective drainage. A dense growth of weeds on the land and enervating malaria are the sure followers of bad drainage.

To avoid this, stringent rules should be enforced to prevent the use of the natural drainage channels for any purpose whatever, but that of conveying away the drainage water that reaches them.

EMPLOYMENT OF LOCAL LABOR.

A great deal of the work, including all the earthwork, could be done by petty contract or day labor, and, in this way, employment could be given, after the harvest is over, to a large number of the residents of the district, who would be willing to do the work.

In what I have above written, I have covered all the points contained in your instructions to me. In conclusion I have to acknowledge the assistance, which as Directors of Tulare Irrigation District, individually and collectively, you have at all times given me.

Respectfully presented,

P. J. FLYNN.

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FLOW OF WATER

IN

IRRIGATION CANALS,

DITCHES, FLUMES, PIPES, SEWERS,

CONDUITS, Etc.

WITH

TABLES

Simplifying and Facilitating the Application of the Formulæ of

KUTTER, D'ARCY AND BAZIN,

BY

P. J. FLYNN, C. E.

Member of the American Society of Civil Engineers; Member of the Technical Society of the Pacific Coast; Late Executive Engineer, Public Works Department, Punjab, India;

AUTHOR OF

"Hydraulic Tables based on Kutter's Formula,"

"Flow of Water in Open Channels," etc.

SAN FRANCISCO, CALIFORNIA.

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LIST OF ILLUSTRATIONS

FLOW OF WATER

IN

IRRIGATION CANALS

AND

Open and Closed Channels Generally.

Article 1. Introduction.

Almost all the old hydraulic formulæ, given below, for finding the mean velocity in open and closed channels have *constant* co-efficients, and are therefore correct for only a small range of channels. They have often been found to give incorrect results with disastrous effects, as on the Rhone, in France, and the Upper Ganges Canal, India. The results of the gauging of large rivers, such as the Mississippi, by Humphrey and Abbott; the Irrawaddy, by Gordon; the Upper Ganges Canal, by Cunningham; small open channels, by Bazin and D'Arcy, and cast-iron pipes by D'Arcy, prove conclusively the inaccuracy of the old formulæ and the accuracy, within certain limits, of the formulæ of Kutter, Bazin and D'Arcy. Ganguillet and Kutter thoroughly investigated the American, French and other experiments, and they gave, as a result of their labors, the formula now generally known as Kutter's formula.

There are so many varying conditions affecting the flow of water, that all hydraulic formulæ are only approximations to the correct result, and the best that an engineer can do is to use the most correct of all the known formulæ.

Major Allan Cunningham, R. E., carried out experiments, on a most extensive scale, lasting over four years, (1874–79), on the Upper Ganges Canal, near Roorkee, India. Major Cunningham states:—*

"The main object of the undertaking was to interpolate something between Mr. Bazin's experiments on small canals and the experiments on American rivers, chiefly with a view to discharge measurement on large canals, the proper measurement of such discharge being of great practical importance, but hitherto attended with much uncertainty. For any such work there are good opportunities in India from its system of canals, both large and small, pre-eminent among which is the Ganges Canal.

'The extensive scale of the operations can be judged from the following abstract:— * * *

"The total number of velocity measurements was about 50,000. Besides these, there were many occasional special experiments, which together form an important addition. * * *

"An important feature in this work is the great range of conditions and data, and therefore of results obtained, this being essential to the discovery of the laws of complex motion. Thus the velocity work was done at thirteen sites, differing much in nature, some being of brick, some of earth; in figure, some being rectangular, some trapezoidal; and in size, the surface-breadth varying from 193 feet to 13 feet, and the central depth from

* Recent Hydraulic Experiments in the Minutes of Proceedings of the Institution of Civil Engineer's, Volume 71.

11 feet to 8 inches. At one of the sites the ranges of some of the conditions and results were: central depth, from 10 feet to 8 inches; surface slope, from 480 to 24 per million; velocity, from 7.7 feet to 0.6 feet per second; cubic discharge, from 7,364 to 114 cubic feet per second. * * *

"After discussing various known formulæ for mean velocity, the only ones that appeared worth extended trial were Bazin's* formulas for the co-efficients β and C, and Kutter's for the co-efficient C. Accordingly, the values of these co-efficients, from the published Tables, have been printed alongside the experimental mean serial values, seventy-six of β and eighty-three of C. As to Bazin's two co-efficients (β, C), the discussion shows that neither is reliable, and that the use of the former with surface-velocity leads to under-estimation of mean velocity, and that the latter is defective in not containing s. As to Kutter's co-efficient C, the discrepancies between the eighty-three experimental and computed values were:—

"Thirteen, over 10 per cent.

"Five, over 7½ per cent.

"Fifteen, over 5 per cent.

"Seventeen, over 3 per cent.

"Thirty-three, under 3 per cent.

"Now in all the discrepancies over 10 per cent., it was found that the state of water was unfavorable for the slope-measurement. Taking this into account, along with the varied evidence in Kutter's work, it seems fair to accept Kutter's co-efficient as of pretty general applicability; also that when the surface slope measurement is good, it will give results seldom exceeding 7½ per cent. error, provided that the rugosity-co-efficient of the

* "Recherches expérimentales sur l'écoulement de l'eau dans les canaux découverts."

formula be known for the site. For practical application extreme care would be necessary about the slope-measurement, and the rugosity-co-efficient could only be determined, according to present knowledge, by special preliminary experiments at each site. * * *

"The accuracy of the D'Arcy-Bazin experiments, on which so much stress had been laid, had never been questioned. The suggestion that the failure of their co-efficients, when applied to the Roorkee results, was due to the disparity of proportions of the D'Arcy-Bazin canals, and the Ganges Canal, was very likely correct, and amounted to an admission of the want of generality of those co-efficients, as urged in the paper. * * * *

"Much special experiment was done (with surface slope measurement), and with the definite result that Kutter's formula was the only one not requiring velocity measurement of pretty general applicability, and would under favorable conditions, give results differing by not more than 7½ per cent., from actual velocity measurements. This was surely a definite and important result."

The above is conclusive as to the correctness of Kutter's formula. For small open channels D'Arcy and Bazin's formulæ, and for cast-iron pipes D'Arcy's formulæ, are generally accepted as being approximately correct. Engineers, who desire to keep up with the progress of Hydraulic Science, now generally use one or all of the formulæ of Kutter, D'Arcy and Bazin, in preference to the old and inaccurate formulæ formerly in universal use. The objection to the former formulæ, however, is that they are in a form not adapted for rapid work, and that they are tedious and troublesome in application.

The object of this work by the writer is to simplify and facilitate the application of these formulæ, so as to

effect a great saving of both time and labor, which is a matter of great importance to an engineer in active practice.

Article 2. The application of Kutter's Formula simplified and facilitated by the use of the Tables.

The plan on which the tables are constructed will be briefly stated here, and their use will be fully explained in Article 13.

The solution of problems, relating to open channels given in this work, is similar to the methods given by the writer in No. 67 of Van Nostrand's Science Series (1883), entitled *Hydraulic Tables based on Kutter's Formula*, and also given in No. 84 (1886), entitled *The Flow of Water in Open Channels, Pipes, Sewers and Conduits.* The present work is based on the same principles, and is intended to facilitate and simplify the computations relating to Open Channels in a somewhat similar way to that already adopted for closed channels.

Kutter's formula for measures in feet is:—

$$v = \left\{ \frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{s}}{1 + \left(41.6 + \frac{.00281}{s}\right) \times \frac{n}{\sqrt{r}}} \right\} \times \sqrt{rs}$$

and putting the first factor on the right hand side of the equation $= c$, we have:—

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s}$$

$$Q = a\,v = c \times a\sqrt{r} \times \sqrt{s}$$

The factors on the right hand side of the equation are tabulated, for different grades and dimensions of channel, and also for different surfaces of channel over which the water flows. The tables give the value of c, $c\sqrt{r}$, a, r, $\sqrt{r}$, $a\sqrt{r}$ and $\sqrt{s}$. All that is then necessary, for the

solution of any problem relating to open channels, is to find out in the tables the value of the factors for the channel under consideration, and to substitute these values in such of the formulæ, 41 to 49, as may be suitable for the work in hand, and then, by simple multiplication and division, the solution of the problem can be quickly obtained.

For example :— Find the velocity in a channel having a bottom width of 18 feet, a depth of 2 feet, side slopes of 1 to 1, a grade of 1 in 1000 and $n = .0275$.

In Table 8 we find under a bed width of 18 feet, and opposite a depth of 2 feet, that $\sqrt{r} = 1.3$. In Table 25, with $n = .0275$, under a slope of 1 in 1000, and opposite $\sqrt{r} = 1.3$, we find the value of $c\sqrt{r} = 73.9$. In Table 33, opposite 1 in 1000, we find $\sqrt{s} = .031623$.

Substituting the values of $c\sqrt{r}$ and $\sqrt{s}$, in formula (41), we have:—

$$v = 73.9 \times .031623 = 2.33 \text{ feet per second.}$$

This is a much quicker method than computing the velocity by working out Kutter's formula (40).

Article 3. Formulæ for Mean Velocity in Open Channels.

In the following formulæ and in what follows:—

v = mean velocity in feet per second.

v_{max} = maximum surface velocity in feet per second.

v_b = bottom velocity in feet per second.

Q = discharge in cubic feet per second.

a = area of cross section of water in square feet.

p = wetted perimeter or length of wetted border in lineal feet.

w = width of surface of water in channel in feet.

$r = \dfrac{a}{p} = \left\{ \begin{array}{l} \text{hydraulic mean depth in feet; = area of} \\ \text{cross section in square feet, divided by} \\ \text{wetted perimeter in lineal feet.} \end{array} \right.$

$r_1 = \dfrac{a}{p + w}$

h = fall of water surface in any distance l.

l = length of water surface for any fall h.

s = fall of water surface (h) in any distance (l) divided by that distance $= \dfrac{h}{l} =$ sine of slope.

f = fall in feet per mile.

c = co-efficient of mean velocity.

$\left. \begin{array}{l} n \\ k \end{array} \right\} =$ the natural co-efficient depending on the nature of the bed; that is, the lining or surface of the channel over which the water flows.

g = acceleration of gravity = 32.16.

The following extract on the value of g is from Merriman's Hydraulics:—

"The symbol g is used in hydraulics to denote the acceleration of gravity; that is, the increase in velocity per second for a body falling freely in a vacuum at the surface of the earth. * * * *

"The following formula of Pierce, which is partly theoretical and partly empirical, gives the value of g in feet for any latitude L, and any elevation e above the sea level, e being taken in feet:—

$$g = 32.0894\,(1 + 0.0052375 \sin^2 L)\,(1 - 0.0000000957e),$$

and from this its value may be computed for any locality.

"For the United States the practical limiting values are

$L = 49°$, $e = 0$; whence $g = 32.186$;

$L = 25°$, $e = 10000$ feet; whence $g = 32.089$.

The value of the acceleration is taken to be, unless otherwise stated,—

$g = 32.16$ feet per second;

from which the frequently recurring quantity $\sqrt{2g}$ is found to be

$$\sqrt{2g} = 8.02.$$

"If greater precision be required, which will rarely be the case, g can be computed from the formula for the particular latitude and elevation above the sea."

The following collection of formulæ, for finding the mean velocity in open channels, is compiled from various authorities. It is believed that such a collection will be useful, not only for reference, but also for comparison of the old with the most modern and accurate formulæ. It is also believed to contain almost all the formulæ in general use at various times and places up to date. All the formulæ are given in feet measures.

D'Aubisson's
Taylor's
Downing's
Beardmore's
Leslie's
Pole's

formula for large and rapid rivers $v = 100\sqrt{rs}$..... (1)

Leslie, for small streams: $v = 68\sqrt{rs}$............. (2)

Stevenson, for streams over 2,000 cubic feet per minute $v = 96\sqrt{rs}$........ (3)

Stevenson, for streams under 2,000 cubic feet per minute $v = 69\sqrt{rs}$........ (4)

D'Aubisson, for velocities over 2 feet per second $v = 95.6\sqrt{rs}$......... (5)

D'Aubisson: $v = (8976.5rs + .012) - .109$......... (6)

Beardmore $v = 94.2\sqrt{rs}$....................... (7)

Eytelwein: $v = 93.4\sqrt{rs}$....................... (8)

Eytelwein: $v = (8975.43\,r\,s + .011589)^{\frac{1}{2}} - .1089$ . . . (9)

Neville, straight rivers with velocity up to 1.5 feet per second $\Big\}$ $v = 92.3\sqrt{rs}$ (10)

Neville, straight rivers with velocity above 1.5 feet per second $\Big\}$ $v = 93.3\sqrt{rs}$ (11)

Neville. $v = 140\sqrt{rs} - 11\sqrt[3]{rs}$. . . . . (12)

Dwyer: $v = 0.92\sqrt{2fr}$ . . . . (13)

This formula corresponds with $v = 94.2\sqrt{rs}$.

Dupuit: $v = \dfrac{sra}{.084w} - (.0067 + 9114rs)^{\frac{1}{2}} - .082$. . . . . . . (14)

Young's formula: $v = \left\{\dfrac{rs}{3A} + \left(\dfrac{B}{12A}\right)^2\right\}^{\frac{1}{2}} - \dfrac{B}{12A}$ . . . . . (15)

where $A =$

$$0.0000001\left(.413 + \frac{1.5625}{r} - \frac{90}{3r+8} - \frac{15}{4r + 0.0296}\right)$$

and $B =$

$$0.0000001\left\{\frac{900r^2}{r^2+0.5} + \frac{1}{3r^4}\left(271.25 + \frac{6.88}{r} + \frac{0.0001146}{r^2}\right)\right\}$$

Young's formula: $v = 84.3\sqrt{rs}$ . . . . (16)

Dubuat's formula

$$v = \frac{88.49\,(\sqrt{r} - 0.03)}{\dfrac{1}{\sqrt{s}} - \text{hyp. log}\left(\dfrac{1}{s} + 1.6\right)^{\frac{1}{2}}} - 0.84\,(\sqrt{r} - .03) \quad \ldots \quad (17)$$

where hyp. log $= 2.302585$.

Girard's formula: $v = (10567.8rs + 2.67)^{\frac{1}{2}} - 1.64$. . . . . (18)

Girard's formula $v = 103\sqrt{rs} - 1.64$. . . . . (19)

De Prony's formula for canals:—

$v = (0.0556 + 10593rs)^{\frac{1}{2}} - 0.2357$. . . . . (20)

For canals and pipes: $v = (0.0237 + 9966rs)^{\frac{1}{2}} - 0.1542$ (21)

Weisbach's coëfficient: $v=(0.00024+8675rs)^{\frac{1}{2}}-0.0154$ (22)

St. Venant's formula: $v=106.068(rs)^{\frac{11}{21}}$ .... (23)

Ellet's formula: $v=0.64\ (\triangle f)^{\frac{1}{2}}+0.04\ \triangle f$...... (24)
where $\triangle$=maxium depth of stream in feet.

Provis's formula: $v=60\ \sqrt{rs}+120\ \sqrt[3]{(rs)^2}$......... (25)

Hagen's formula: $v=4.39\ (r)^{\frac{1}{2}}\times(s)^{\frac{1}{6}}$.............. (26)

Schlichting's derivation of Hagen's formula:—

For large rivers and canals: $v=6\ (r)^{\frac{1}{2}}\times(s)^{\frac{1}{4}}$........ (27)

Ganguillet and Kutter condemn Hagen's formulæ as "absolutely useless."

Fanning's formula: $v=\left(\frac{2grs}{m}\right)^{\frac{1}{2}}$.................. (28)

and $m=\frac{2grs}{v^2}$

Humphrey's and Abbot's formula:—

$$v=\left\{\sqrt{.0081b+(225r_1s^{\frac{1}{2}})^{\frac{1}{2}}}-.09b^{\frac{1}{2}}\right\}^2-\frac{2.4\ (v^1)^{\frac{1}{2}}}{1+p}\ \ldots\ldots \quad (29)$$

Where b =function of depth for small streams $=\frac{1.69}{(r+1.5)^{\frac{1}{2}}}$

and v^1= value of first term in expression for v.

For rivers whose hydraulic mean depth exceeds 12 or 15 feet, b may be assumed to be 0.1856, which will make the numerical value of the term involving b so small that it may be generally neglected, reducing the above equation to

$$v=\left\{(225r_1\,s^{\frac{1}{2}})^{\frac{1}{4}}-.0388\right\}^2\ \ldots\ldots \quad (30)$$

Gauchler's formulæ:

When s is greater than .0007, that is greater than 1 in 1429,

$$(v)^{\frac{1}{2}}=1.219\times k_1\times r^{\frac{1}{3}}\times s^{\frac{1}{4}}\ \ldots\ldots \quad (31)$$

When s is less than .0007, that is less than 1 in 1429

$$(v)^{\frac{1}{4}}=\frac{k_2}{1.104}\times r^{\frac{1}{3}}\times s^{\frac{1}{2}}\ \ldots\ldots \quad (32)$$

TABLE 1. Giving the values of the co-efficients, k_1, k_2, to be employed in Gauchler's formulæ for canals and rivers and other open channels:

NATURE OF CHANNEL.	k_1 s greater than .0007	k_2 s less than .0007
Masonry, cut stone and mortar...	From 8.5 to 10	From 8.5 to 9.0
Good masonry....................	" 7.6 " 8.5	" 8.0 " 8.5
Masonry sides; earth bottom.....	" 6.6 " 7.6	" 7.7 " 8.0
Small water-courses in earth free of weeds......................	" 5.7 " 6.7	" 7.0 " 7.7
Small water-courses in earth, grass on slopes......................	" 5.0 " 5.7	" 6.6 " 7.0
Rivers..........................	Nil	" 6.3 " 7.0

Moleswort's formula:

$$v = \sqrt{krs} \qquad (33)$$

TABLE 2. Giving the value of the co-efficients k in Molesworth's formula for canals and rivers.

NATURE OF CHANNEL.	VALUES OF k FOR VELOCITIES.	
	Less than 4 feet per second.	More than 4 feet per second.
Brickwork	8800	8500
Earth	7200	6800
Shingle	6400	5900
Rough, with bowlders....	5300	4700

In very large channels, rivers, etc., the description of the channel affects the result so slightly that it may be practically neglected, and assumed =from 8,500 to 9,000.

Bazin's formulæ:

For very even surfaces, fine plastered sides and bed, planed planks, etc :

$$v = \sqrt{1 \div .0000045\left(10.16 + \frac{1}{r}\right)} \times \sqrt{rs} \qquad (34)$$

For even surfaces, such as cut stone, brickwork, unplaned planking, mortar, etc.:

$$v = \sqrt{1 \div .000013\left(4.354+\frac{1}{r}\right)} \cdot \sqrt{rs} \ldots\ldots\ldots\ldots (35)$$

For slightly uneven surfaces, such as rubble masonry:

$$v = \sqrt{1 \div .00006\left(1.219 + \frac{1}{r}\right)} \times \sqrt{rs} \ldots\ldots\ldots\ldots (36)$$

For uneven surfaces, such as earth:

$$v = \sqrt{1 \div .00035\left(0.2438 + \frac{1}{r}\right)} \cdot \sqrt{rs} \ldots\ldots\ldots\ldots (37)$$

A modification of Bazin's formula (37), known as D'Arcy Bazin's:

$$v = r\sqrt{\frac{1000s}{.08534r + 0.35}} \ldots\ldots\ldots\ldots (38)$$

Brandreth's modification of Bazin's formula (37) is:

$$v = \frac{[illegible]}{\sqrt{7}} \cdot \frac{2r}{1.7066r} \times \sqrt{f} \ldots\ldots\ldots\ldots (39)$$

where f = fall in 5,000 feet, which is the length of the old English mile, now used on Indian irrigation canals.

Kutter's formula is:

$$v = \left(\frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{s}}{1 + \left(41.6 + \frac{.00281}{s}\right)\frac{n}{\sqrt{r}}}\right) \times \sqrt{rs} \ldots\ldots (40)$$

and calling the first term of the right-hand side of the equation equal c, we have Chezy's formula

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} \ldots\ldots (41)$$

$$[illegible] \ldots\ldots (42)$$

$$\sqrt{s}=\frac{v}{c\sqrt{r}} \quad \text{.......... (43)}$$

$$s=\left(\frac{v}{c\sqrt{r}}\right)^2 \quad \text{.......... (44)}$$

$$\left.\begin{aligned}\text{Now } Q&=av=a\times c\sqrt{r}\times\sqrt{s}\\&=a\sqrt{r}\times c\times\sqrt{s}\end{aligned}\right\} \quad \text{.......... (45)}$$

$$\therefore\ a=\frac{Q}{v} \quad \text{.......... (46)}$$

$$ac\sqrt{r}=\frac{Q}{\sqrt{s}} \quad \text{.......... (47)}$$

$$\sqrt{s}=\frac{Q}{ac\sqrt{r}} \quad \text{.......... (48)}$$

$$s=\left(\frac{Q}{ac\sqrt{r}}\right)^2 \quad \text{.......... (49)}$$

$$c=\frac{V}{\sqrt{r}\times\sqrt{s}}=\frac{Q}{a\sqrt{r}\times\sqrt{s}} \quad \text{.......... (50)}$$

Article 4. Remarks on the Formulæ.

Most of the old formulæ have *constant* co-efficients, and therefore give accurate results for only one channel, having a hydraulic mean radius of a certain value. Only four of the authorities, whose formula are given in Article 3, have taken into account the nature of the material forming the surface of the channel. These are Gauchler, Bazin, Molesworth and Kutter. The value of the co-efficients in Bazin's formulæ depends on the nature of the surface of the material over which the water flows, and also the hydraulic mean depth. These co-efficients are not affected by the slope.

For small channels of less than 20 feet bed Bazin's formula, for earthen channels in good order, gives very fair results, and tables based on it have been used by the Irrigation Departments in Northern India, for com-

puting the velocities in the distributing channels (rajbuhas), but Kutter's formula is superseding it there, as in almost all other countries where its accuracy has been thoroughly investigated.

The formulæ of Gauchler, Molesworth and Kutter have varying co-efficients, which depend for their value on three things :—

The hydraulic mean depth,

The slope or grade of bed, and

The nature of the surface of the material, or the wetted peimeter, over which the water flows.

The following table shows the value of c, in Kutter's formula, for a wide range of channels in earth, that will cover anything likely to occur in the ordinary practice of an engineer.

TABLE 3. Values of c for earthen channels by Kutter's formula.

Slope 1 in	$n = .0225$					$n = .035$				
	$\sqrt{r}$ in feet.					$\sqrt{r}$ in feet.				
	0.4	1.0	1.8	2.5	4.0	0.4	1.0	1.8	2.5	4.0
	c	c	c	c	c	c	c	c	c	c
1000	35.7	62.5	80.3	89.2	99.9	19.7	37.6	51.6	59.3	69.2
1250	35.5	62.3	80.3	89.3	100.2	19.6	37.6	51.6	59.4	69.4
1667	35.2	62.1	80.3	89.5	100.6	19.4	37.4	51.6	59.5	69.8
2500	34.6	61.7	80.3	89.8	101.4	19.1	37.1	51.6	59.7	70.4
3333	34.	61.2	80.2	90.1	102.2	18.8	36.9	51.6	59.9	71.0
5000	33.	60.5	80.3	90.7	103.7	18.3	36.4	51.6	60.4	72.2
7500	31.6	59.4	80.3	91.5	106.0	17.6	35.8	51.6	60.9	73.9
10000	30.5	58.5	80.3	92.3	107.9	17.1	35.3	51.6	60.5	75.4
15840	28.5	56.7	80.2	93.9	112.2	16.2	34.3	51.6	62.5	78.6
20000	27.4	55.7	80.2	94.8	115.0	15.6	33.8	51.5	63.1	80.6

An inspection of the tables will show the difference in the value of c, caused by the difference in slope, and also of hydraulic mean depth. It shows, for instance, that

with $n = .0225$ and a slope of 1 in 1000 the value of c corresponding to $\sqrt{r} = 0.4$ is 35.7, while the value of c corresponding to $\sqrt{r} = 4.0$ is 99.9, or an increase of about 180 per cent. By the old formulæ the channel, with the small hydraulic mean depth, would have the same co-efficient as the larger channel, and would therefore give very inaccurate results.

We also see that with the same $\sqrt{r} = 4.0$, the value of c for a slope of 1 in 1000 is 99.9, and the value of c for a slope of 1 in 20000 is 115.0, or an *increase* of about 15 per cent., while with $\sqrt{r} = 0.4$ there is a *decrease* from 35.7 to 27.4, or a decrease of about 23 per cent.

We again find that when $\sqrt{r} = 1.8$ (which is the nearest value of $\sqrt{r}$ to 1.811) the co-efficients for the same value of n are the same for all slopes; that when $\sqrt{r}$ has a less value than 1.8 the co-efficients increase with an *increase* of slope, and when $\sqrt{r}$ has a greater value than 1.8 the co-efficients increase with the *decrease of* slope.

D'Arcy's formulæ will be referred to in the Article on the *Flow of Water in Pipes*, etc.

Article 5. Bazin's Formula for Channels in Earth.

For small channels in earth in moderately good order Bazin's formula (37) gives a tolerable close approximation to the mean velocity.

Table 30, giving mean velocities and discharges up to 20 feet bed width, was computed for the Punjab Irrigation Department by Captain Allen Cunningham, R. E. The table was computed by a modification of Bazin's formula (37) given by Captain A. B. Brandreth, R. E., *for channels whose beds and sides are of earth.**

* Volumes 4 and 5 of Second Series of Professional Papers on Indian Engineering.

This modified form formula (39) was adopted, as it was better suited for computation of tables than Bazin's formula (37). This formula is:—

$$v = \frac{2r}{\sqrt{7+1.7066\,r}} \times \sqrt{f}$$

where f is the fall in 5,000 feet, in feet. The length of the old English mile now used on Indian canals is 5,000 feet.

In order to show how the modified form of Bazin is derived, it is given below.

Bazin's formula for earthen channels is:—

$$v = \sqrt{\frac{1}{.00035 + \left(.2438 + \frac{1}{r}\right)}} \times \sqrt{rs}$$

but $\sqrt{rs} = \sqrt{r} \times \sqrt{\frac{f}{5000}} = 2\sqrt{r} \times \sqrt{\frac{f}{20000}}$

substitute this value of $\sqrt{rs}$ and reduce and

$$v = \sqrt{\frac{1}{.000085333\,r + .00035}} \times 2\,r \times \sqrt{\frac{f}{20000}}$$

$$\therefore v = \frac{2\,r}{\sqrt{7 + 1.7066\,r}} \times \sqrt{f}$$

Article 6. Comparing Kutter's and Bazin's Formulæ.

The following table gives a comparison between the results obtained by Bazin's formula (37) for earthen channels, and Kutter's formula with $n = .025$ and $n = .0275$, and it shows that, for the given channels, Bazin's formula agrees very nearly with Kutter $n = .0275$ up to 3 feet in depth, and with Kutter $n = .025$ from 3 to 5 feet in depth. It is also shown that Bazin's formula is almost a mean between Kutter with $n = .025$,

and $n = .0275$; that is, that it almost suits *canals and rivers in earth of tolerably uniform cross-section, slope and direction, in moderately good average order and regimen and free from stones and weeds*, and also *canals and rivers in earth below the average in order and regimen.*

The results again show that it gives too low a velocity for *canals in earth above the average in order and regimen* with $n = .0225$ or a less value of n, and it further shows that it gives too high a velocity for *canals and rivers in earth, in rather bad order and regimen, having stones and weeds occasionally*, and obstructed *by detritus*, $n = .030$.

Bazin's formula (37) gives correct results for earthen channels with only one value of n, while Kutter's formula is suited to any channel having either a very rough, medium or very smooth surface.

TABLE 4. Giving the velocity and discharge of earthen channels according to the formulæ of Bazin and also Kutter.

v = mean velocity in feet per second.
Q = discharge in cubic feet per second.

Bed width 10 feet. Side slopes 1 to 1. Slope 1 in 2500. $\sqrt{s} = .02$.

Depth in feet.	Area in square feet, a	Hydraulic mean depth in feet, r . . .	$\sqrt{r}$	Bazin, for Earthen Channels.		Kutter, $n = .025$		Kutter, $n = .0275$	
				v	Q	v	Q	v	Q
1.0	11.00	0.858	0.93	0.83	9.17	0.97	10.67	0.87	9.57
1.5	17.25	1.211	1.10	1.14	19.63	1.26	21.73	1.14	19.66
2.0	24.00	1.533	1.24	1.40	33.56	1.51	36.24	1.36	32.64
2.5	31.25	1.831	1.35	1.63	50.85	1.72	53.75	1.56	48.75
3.0	39.00	2.110	1.45	1.83	71.48	1.91	74.49	1.73	67.47
3.5	47.25	2.375	1.54	2.02	95.46	2.08	98.28	1.89	89.30
4.0	56.00	2.628	1.62	2.19	122.81	2.24	125.44	2.03	113.68
4.5	65.25	2.871	1.70	2.35	153.61	2.39	155.95	2.17	141.59
5.0	75.00	3.107	1.76	2.51	187.91	2.53	189.75	2.29	171.75

Bed width 20 feet. Side slope 1 to 1. Slope 1 in 2500. $\sqrt{s} = .02$.

Depth in feet.	Area in square feet, a......	Hydraulic mean depth in feet, r...	$\sqrt{r}$	Bazin, for Earthen Channels.		Kutter, $n = .025$		Kutter, $n = .0275$	
				v	Q	v	Q	v	Q
1.0	21.00	0.920	0.96	0.89	18.66	1.02	21.42	0.91	19.11
1.5	32.25	1.330	1.15	1.24	39.85	1.36	43.86	1.22	39.34
2.0	44.00	1.715	1.31	1.54	67.74	1.64	72.16	1.48	65.12
2.5	56.25	2.078	1.44	1.81	101.80	1.89	106.31	1.71	96.19
3.0	69.00	2.422	1.55	2.05	141.68	2.12	146.28	1.91	131.79
3.5	82.25	2.751	1.66	2.28	187.15	2.32	190.82	2.10	172.72
4.0	96.00	3.066	1.75	2.48	238.03	2.50	240.00	2.27	217.92
4.5	110.25	3.369	1.83	2.67	294.21	2.67	294.37	2.43	267.91
5.0	125.00	3.661	1.91	2.85	555.64	2.83	353.75	2.58	322.50

Article 7 Value of n.

The accuracy of Kutter's formula depends, in a great measure, on the proper selection of the co-efficient of roughness n. Experience is required in order to give the right value to this co-efficient, and, to this end, great assistance can be obtained in making this selection, by consulting and comparing the results obtained from experiments on the flow of water already made in different channels.

In some cases it would be well to provide for the contingency of future deterioration of channel, by selecting a high value of n, as, for instance, where a dense growth of weeds is likely to occur in small channels, and also where channels are likely not to be kept in a state of good repair.

Table 5, giving the value of n for different materials, is compiled from Kutter, Jackson and Hering, and this value of n applies also, in each instance, to the surfaces of other material equally rough.

Table 5. Giving the value of n for different channels.

$n = .009$, well-planed timber, in perfect order and alignment; otherwise, perhaps .01 would be suitable.

$n = .010$, plaster in pure cement: planed timber; glazed, coated, or enamelled stoneware and iron pipes; glazed surfaces of every sort in perfect order.

$n = .011$, plaster in cement with one-third sand in good condition; also for iron, cement, and terra-cotta pipes, well joined and in best order.

$n = .012$, unplaned timber, when perfectly continuous on the inside; flumes.

$n = .013$, ashlar and well-laid brickwork; ordinary metal; earthenware and stoneware pipe in good condition, but not new; cement and terra-cotta pipe not well jointed nor in perfect order; plaster and planed wood in imperfect or inferior condition; and, generally, the materials mentioned with $n = .010$, when in imperfect or inferior condition.

$n = .015$, second-class or rough-faced brickwork; well-dressed stonework; foul and slightly tuberculated iron; cement and terra-cotta pipes, with imperfect joints and in bad order; and canvas lining on wooden frames.

$n = .017$, brickwork, ashlar, and stoneware in an inferior condition; tuberculated iron pipes; rubble in cement or plaster in good order; fine gravel, well rammed, $\frac{1}{3}$ to $\frac{2}{3}$ inches diameter; and, generally, the materials mentioned with $n = .013$ when in bad order and condition.

$n = .020$, rubble in cement in an inferior condition; coarse rubble, rough-set in a normal condition; coarse rubble set dry; ruined brickwork and masonry; coarse gravel, well rammed, from 1 to 1½ inch diameter; canals with beds and banks of very firm, regular gravel, carefully trimmed and rammed in defective places; rough rubble, with bed partially covered with silt and mud; rectangular wooden troughs, with battens on the inside two inches apart; trimmed earth in perfect order.

$n = .0225$, canals in earth above the average in order and regimen.

$n = .025$, canals and rivers in earth of tolerably uniform cross-section, slope, and direction, in moderately good order and regimen, and free from stones and weeds.

$n = .0275$, canals and rivers in earth below the average in order and regimen.

$n = .030$, canals and rivers in earth in rather bad order and regimen, having stones and weeds occasionally, and obstructed by detritus.

$n = .035$, suitable for rivers and canals with earthen beds in bad order and regimen, and having stones and weeds in great quantities.

$n = .05$, torrents encumbered with detritus.

TABLE 5 (continued). The following table, giving values of n for different surfaces exposed to the flow of water, is taken from Jackson's translation of Kutter. The dimensions are, however, changed from metrical to feet measures.

$r =$ hydraulic mean depth in feet.

$s =$ sine of slope.

No.	SERIES OF BAZIN.	r in feet	s	Breadth at water surface in feet	Depth in feet.	n
28	Carefully planed plank...	0.07	0.0048922	0.328	0.14	0.0096
29	" " "	0.05	0.0152370	0.328	0.079	0.0087
24	In cement—semi-circular.	0.82	0.0014243	3.28	1.47	0.01005
2	" rectangular..	0.49	0.005060	5.9	0.59	0.01040
25	In cement, with one-third sand—semi-circular...	0.85	0.0013802	3.28	1.61	0.01113
26	Plank—semi-circular....	0.91	0.0015227	3.6	1.61	0.01195
21	" trapezoidal......	0.82	0.0015213	4.6	1.24	0.01255
22	" "	0.65	0.0048751	4.36	0.98	0.01190
23	" triangular 45°....	0.65	0.004655	4.36	1.87	0.0119
6	" rectangular......	0.65	0.0022136	6.5	0.85	0.13
7	" "	0.52	0.004889	6.5	0.62	0.0119
8	" "	0.46	0.0081629	6.5	0.52	0.0115
9	" "	0.72	0.0014678	6.5	0.91	0.0129
10	" "	0.46	0.0058744	6.5	0.55	0.0117
11	" "	0.42	0.0083805	6.5	0.49	0.0114
18	" "	0.65	0.0045988	3.9	0.91	0.0114
19	" "	0.49	0.0042731	2.6	0.82	0.0114
20	" "	0.32	0.0059829	1.6	0.62	0.0114
	RAMMED GRAVEL					
27	$\frac{2}{5}$ to $\frac{4}{5}$ inches thick—semi-circular...............	0.75	0.0013639	3.28	1.34	0.0163
4	$\frac{2}{5}$ to $\frac{4}{5}$ inches thick—rectangular	0.65	0.0049736	6.0	0.85	0.0170
	BATTENS PLACED					
12	$\frac{2}{5}$ inch apart—rectangular	0.75	0.0014678	6.4	1.01	0.0149
13	" " "	0.55	0.0059664	6.4	0.65	0.0147
14	" " "	0.49	0.0088618	6.4	0.59	0.0149
15	2 inches " "	0.95	0.0014678	6.4	1.31	0.0208
16	" " "	0.69	0.0059976	6.4	0.88	0.0211
17	" " "	0.63	0.0088618	6.4	0.78	0.0215
1.2	Ashlar—rectangular......	1.77	0.0008400	8.5	3.0	0.0133
3	Brickwork—rectangular..	0.55	0.0050250	3.0	0.65	0.0129
39	Ashlar..................	0.59	0.0081	3.9	0.85	0.0129
	RUBBLE,					
32	Rather damaged—rectangular................	0.52	0.10076	5.9	0.63	0.0167

TABLE 5.—(Continued.)

No.	SERIES OF BAZIN.	r in feet.	s	Breadth at water surface in feet	Depth in feet.	n
	RUBBLE,					
33	Rather damaged—rectangular	0.65	0.036856	5.9	0.88	0.0170
1.4	Rather damaged—new....	0.63	0.060	3.28	0.95	0.0180
1.3	" " "	0.72	0.029	3.28	1.18	0.0184
1.6	" " "	0.82	0.014	3.28	1.54	0.0182
1.5	" " "	0.88	0.0122	3.28	1.60	0.0192
44	With deposits on the bed, rectangular	1.47	0.00032	6.56	2.62	0.0204
46	With deposits on the bed, rectangular	1.31	0.00032	6.56	2.29	0.0210
35	Damaged rubble — trapezoidal	1.21	0.014221	4.9	2.29	0.0220
	OTHER OBSERVATIONS					
	Gontenbachschale, new rubble—semi-circular..	0.32	0.044	5.5	0.59	0.0145
	G'rumbachschale semi-circular, damaged......	0.46	0.09927	8.5	0.82	0.0175
	Gerbebachschale — semi-circular, damaged......	0.19	0.168	3.7	0.29	0.0185
	Alpbachschale — semi-circular, much damaged..	0.72	0.0274	8.2	1.18	0.0230
	Marseilles Canal.........	2.87	0.00043	19.6	4.4	0.0244
	Jard Canal..............	1.97	0.0004	19.6	4.4	0.0255
	Chesapeake Ohio Canal..	3.7	0.000698	22.6	7.9	0.033
	Canal in England........	2.43	0.000063	17.7	3.9	0.0184
	Lanter Canal, at Newbury	1.81	0.000664	29.5	1.8	0.0262
	Pannerden Canal, in Holland..................	10.2	0.000224	558.	9.8	0.0254
	Canal of Marmels........	2.31	0.0005	26.2	2.6	0.0301
	Linth Canal.............	7.8	0.00034	123.	10.8	0.0222
	Hübengraben...........	0.6	0.0013	4.8	0.8	0.0237
	Hockenbach............	0.87	0.000787	11.1	1.1	0.0243
	Speyerbach.............	1.46	0.000667	16.4	1.9	0.0260
	Mississippi..............	65.6	0.000667	2493.	16.4	0.0270
	Bayou Plaquemine.......	16.8	0.00017	275.	25.5	0.0294
	Bayou La Fourche.......	13.1	0.00004	220.	23.5	0.0200
	Ohio, Point Pleasant.....	6.7	0.000093	1066.	7.9	0.0210
	Tiber, at Rome..........	9.4	0.00013	239.	14.8	0.0228
	Newka...................	17.4	0.000015	886.	21	0.0252
	Newa....................	35.4	0.000014	1214.	19.7	0.0262
	Weser..................	9.5	0.0002	394.	9.8	0.0232
	Elbe....................	10.9	0.00031	315.	43.6	0.0285
	Rhine, in Holland.......	12.4	0.00015	1312.	14.7	0.0243
	Seine, at Paris..........	12.1	0.000137	......	......	0.025
	Seine, at Poissy.........	13.4	0.00007	......	......	0.026
	Saone, at Raconnay......	11.8	0.00004	......	......	0.028
	Haine..................	5.2	0.0001	......	......	0.026

TABLE 5.—(Continued.)

SERIES OF BAZIN.	r in feet.	s	Breadth at water surface in feet.	Depth in feet.	n
CHANNELS OBSTRUCTED BY DETRITUS.					
Rhine, at Speyer	9.7	0.000112	1440	9.7	0.026
Rhine, at Germersheim	10.8	0.000247	748	......	0.0227
Rhine, at Basle	6.9	0.001218	660	9.1	0.03
Lech	3.1	0.00115	157	3.8	0.022
Saalach	1.4	0.0011	68	2.1	0.027
Salzach	4.1	0.0012	38	11.8	0.028
Issar	3.9	0.0025	164	4.4	0.0305
Escher Canal	4.0	0.003	72	4.9	0.03
Plessur	3.5	0.00965	42	4.6	0.027
Rhine, at Rhinewald	.79	0.0142	14	.99	0.031
Mösa, at Misox	1.2	0.01187	13	1.3	0.031
Rhine, at Domleschgerthal	1.9	0.0075	16	2.4	0.035
Simme, at Leuk	1.6	0.0105	....	......	0.0345

In order to show to what extent the value of n affects the velocity and discharge of channels, two examples are given in table 6.

TABLE 6. Showing the effect of the co-efficient of roughness n on the velocity in channels.

Value of n	Bed width in feet.	Depth in feet.	Side Slopes.	Grade in feet per mile.	Mean velocity in feet per second.	Discharge in cubic feet per second.
.0225	10	2	1 to 1	8	3.32	79.7
.025	10	2	"	8	2.96	71.0
.0275	10	2	"	8	2.67	64.1
.03	10	2	"	8	2.43	58.3
.035	10	2	"	8	2.05	49.2
.0225	80	5	1½ to 1	2	3.49	1527.
.025	80	5	"	2	3.15	1378.
.0275	80	5	"	2	2.87	1256.
.03	80	5	"	2	2.64	1155.
.035	80	5	"	2	2.28	998.

In the first channel with a bed-width of 10 feet, the difference in results shows that with a value of $n = .0225$ the channel has a discharge of over 60 per cent. more than when its value of $n = .035$. This shows the great necessity of keeping small irrigation channels clear of

sand bars, brush, weeds, grass and other obstructions to the flow.

Again, in the larger channel with a bed-width of 80 feet, the difference in results, obtained from the highest and lowest values of n, given in table 6, shows a variation of over 53 per cent. in the velocity and discharge. It is shown that the smaller the channel the greater is the percentage of loss by keeping it in a bad state of repair.

Article 8. Side Slopes.

Tables 8, 9, 11 and 13 are computed for channels having side slopes of 1 to 1, ½ to 1, 1½ to 1, and vertical or rectangular.

When the bed width is greater than 60 feet, the side slopes have very little effect on the velocity. Table 7, given below, well exemplifies this. Six channels are given, with varying bed widths, depths and grades, and each channel has five different side slopes. On inspection, it will be seen that the change in the side slope makes no appreciable change in the velocity so long as the bed width, depth and grade or longitudinal slope remains the same. For instance, with a bed width of 70 feet, a depth of 1 foot, and a slope of 1 in 5000, the mean velocity is 0.74 feet per second for the five side slopes. Again, with a bed width of 300 feet, a depth of 14 feet, and a grade of 1 in 20,000, the mean velocity varies so little that it is substantially the same for the five channels, the greatest velocity being 2.35 feet per second, and the least velocity 2.32 feet per second. The table shows, however, that the discharge is increased with the increased flatness of the slopes.

In Table 8, with side slopes of 1 to 1, the values of the factors a, $\sqrt{r}$ and $a\sqrt{r}$ are given for channels up to

a bed width of 300 feet. In Tables 9, 11 and 13, the values of these factors are given only for channels up to a bed width of 60 feet. For all channels having a greater bed width than 60 feet, and side slopes different from 1 to 1, the velocity can be found for a channel with the same bed width, but with side slopes of 1 to 1, and this will be the velocity required. To find the discharge, this velocity can be multipled by the area of channel. For example, let the velocity and discharge be required for a channel having a bed width of 160 feet, depth of 10 feet, a grade of 1 in 15,840, or 4 inches per mile, and with $n = .025$, and side slopes of $1\frac{1}{2}$ to 1. As the tables do not give the value of the factors for a channel of these dimensions with side slopes $1\frac{1}{2}$ to 1, let us look out, in Table 8, the value of $\sqrt{r}$ for a similar channel, but with side slopes of 1 to 1, and we find that it is equal to 3.005. Now the actual value of $\sqrt{r}$ for a side slope of $1\frac{1}{2}$ to 1 is equal to 2.988, so that, practically, the value given in Table 8 is correct.

Now, working out the velocities, we find that side slopes of 1 to 1 give a mean velocity of 2.09 feet per second, and side slopes of $1\frac{1}{2}$ to 1 give a velocity of 2.08 feet per second, as shown in Table 7.

The discharge, however, is increased in proportion to the increase of area of the channel by the increased flatness of the slopes. This is shown by the last column of Table 7, showing the discharge of the channels. In the instance just given, Table 7 shows that with side slopes of 1 to 1 the discharge is 3553.7 cubic feet per second, but with side slopes of $1\frac{1}{2}$ to 1 the discharge is 3631.3 cubic feet per second.

TABLE 7. Showing the velocity and discharge of channels having different side slopes. $n = .025$.

Bed 70 feet. Depth 1 foot. Slope 1 in 5,000. $n = .025$.

Cross Section	a	r	$\sqrt{r}$	$c\sqrt{r}$	$\sqrt{s}$	Velocity in feet per second.	Discharge in cubic feet per second.
Rectangular	70.0	0.972	0.986	52.438	.014142	0.7415	51.91
½ to 1.....	70.5	.976	.988	52.604	.014142	0.744	52.45
1 to 1.....	71.0	.975	.987	52.521	.014142	0.743	52.75
1½ to 1.....	71.5	.971	.986	52.438	.014142	0.742	53.05
2 to 1.....	72.0	.969	.983	52.189	.014142	0.738	53.14

Bed 70 feet. Depth 6 feet. Slope 1 in 5,000. $n = .025$.

Cross Section	a	r	$\sqrt{r}$	$c\sqrt{r}$	$\sqrt{s}$	Velocity in feet per second.	Discharge in cubic feet per second.
Rectangular	420	5.122	2.263	179.504	.014142	2.5385	1066.2
½ to 1.....	438	5.258	2.293	182.744	.014142	2.5844	1132.0
1 to 1.....	456	5.243	2.289	182.312	.014142	2.5782	1175.7
1½ to 1.....	474	5.172	2.270	180.260	.014142	2.5492	1208.3
2 to 1.....	492	5.081	2.254	178.532	.014142	2.5248	1242.2

Bed 160 feet. Depth 2 feet. Slope 1 in 15,840. $n = .025$.

Cross Section	a	r	$\sqrt{r}$	$c\sqrt{r}$	$\sqrt{s}$	Velocity in feet per second.	Discharge in cubic feet per second.
Rectangular	320	1.951	1.397	89.715	.007946	0.7129	228.1
½ to 1.....	322	1.958	1.413	91.074	.007946	0.7237	233.0
1 to 1.....	324	1.956	1.398	89.810	.007946	0.7136	231.2
1½ to 1.....	326	1.950	1.396	89.620	.007946	0.7121	232.1
2 to 1.....	328	1.942	1.393	89.335	.007946	0.7099	232.8

Bed 160 feet. Depth 10 feet. Slope 1 in 15,840. $n = .025$.

Cross Section	a	r	$\sqrt{r}$	$c\sqrt{r}$	$\sqrt{s}$	Velocity in feet per second.	Discharge in cubic feet per second
Rectangular	1600	8.889	2.981	260.334	.007946	2.0686	3309.8
½ to 1	1650	9.048	3.008	263.420	.007946	2.0931	3453.6
1 to 1	1700	9.029	3.005	263.075	.007946	2.0904	3553.7
1½ to 1.....	1750	8.926	2.988	261.132	.007946	2.0750	3631.3
2 to 1.....	1800	8.793	2.965	258.510	.007946	2.0541	3697.4

Bed 300 feet. Depth 2 feet. Slope 1 in 20,000. $n = .025$.

Cross Section	a	r	$\sqrt{r}$	$c\sqrt{r}$	$\sqrt{s}$	Velocity in feet per second.	Discharge in cubic feet per second.
Rectangular	600	1.974	1.405	90.490	.007071	0.6399	383.9
½ to 1.....	602	1.977	1.406	90.588	.007071	0.6405	385.6
1 to 1.....	604	1.976	1.405	90.490	.007071	0 6399	386.5
1½ to 1	606	1.973	1.404	90.384	.007071	0.6391	387.3
2 to 1.....	608	1.968	1.403	90.294	.007071	0.6385	388.2

Bed 300 feet. Depth 14 feet. Slope 1 in 20,000 $n = .025$.

Cross Section	a	r	$\sqrt{r}$	$c\sqrt{r}$	$\sqrt{s}$	Velocity in feet per second.	Discharge in cubic feet per second.
Rectangular	4200	12.835	3.583	330.594	.007071	2.3376	9818
½ to 1....	4298	12.973	3.600	332.600	.007071	2.3518	10108
1 to 1.....	4396	12.940	3.597	332.246	.007071	2.3493	10328
1½ to 1.....	4494	12.823	3.581	330.358	.007071	2.3360	10498
2 to 1....	4592	12.664	3.560	327.880	.007071	2.3184	10646

Article 9. Open Channels Having the Same Velocity.

Channels having the same slope, the same value of n, and also the same value of $\sqrt{r}$, have the same velocity. For example, a channel 70 feet wide on bottom, depth of water 4 feet, side slopes 1 to 1, grade 1 in 1,544, and $n = .03$, has a mean velocity of 2.98 feet per second. The $\sqrt{r}$ of this channel will be found in Table 8, = 1.9, and if we examine this table we find channels of the following dimensions that have the same value of $\sqrt{r}$, and therefore the same velocity.

Bed 70 feet,	depth	4	feet,	$\sqrt{r} = 1.9$
Bed 45 "	"	4.25	"	$\sqrt{r} = 1.9$
Bed 25 "	"	4.75	"	$\sqrt{r} = 1.9$
Bed 20 "	"	5	"	$\sqrt{r} = 1.9$
Bed 14 "	"	5.50	"	$\sqrt{r} = 1.9$

These five channels have the same velocity. They have, however, different discharges, varying with the area of each channel. Channels having the same velocity can also be found having side slopes of ½ to 1 and 1½ to 1, and also rectangular in section.

Article 10. Open Equivalent Discharging Channels.

Channels having the same, or nearly the same, value of $\sqrt{r}$ and a have the same discharging capacity. For example, a channel having a bed width of 12 feet, depth 3 feet, side slopes of 1 to 1, a grade of 5 feet per mile, and $n = .0275$, has a discharge of 123.75 cubic feet per second. Now all channels with the same area, 45 square feet, and the same value of $\sqrt{r} = 1.482$, will have the same discharge when s and n are the same. An inspection of Table 8, with side slopes of 1 to 1, will show channels of a nearly equivalent discharge, thus:—

Bed 10 feet, depth 3.25 feet, $\sqrt{r} = 1.498$, $a = 43$

Bed 15 " " 2.75 " $\sqrt{r} = 1.464$, $a = 48.8$

Again, a depth or width being first given, the corresponding width or depth to give the required discharge can be found after a few trials.

Article 11. Interpolating.

In tables 15 to 27 inclusive, there is given a column headed "diff.", which gives the differences of c and $c\sqrt{r}$, equivalent to a difference of value $= .01$ in $\sqrt{r}$, and this column will be found useful in interpolating values of c and $c\sqrt{r}$ between those given in the tables. For instance, we have a channel which has a grade of 1 in 1000, its value of $n = .02$ and $\sqrt{r} = 1.44$, and we want the value of c corresponding to this value of $\sqrt{r}$.

In Table 22, $n = .02$ and under a slope of 1 in 1000, the nearest value to $\sqrt{r} = 1.44$ that we find is 1.4, and the value of c opposite this 82.6. The column of differences shows that for a value of $\sqrt{r} = .01$ the corresponding value of $c = .22$, therefore $.22 \times 4 = .88$ has to be added to 87.6 thus:—

$\sqrt{r} = 1.4$ and corresponding value of $c = 82.6$

$\sqrt{r} = 0.04$ and corresponding valve of $c = \underline{\ \ 0.88}$

$\therefore \sqrt{r} = 1.44$ and corresponding value of $c = 83.48$

Frequently in the examples, in order to avoid long explanations, it is proposed to find the value of c or $c\sqrt{r}$, equivalent to a value of $\sqrt{r}$ not given in the tables, but it is understood that the method of interpolation, just explained, is intended to be used to find the values of c and $c\sqrt{r}$.

Article 12. Preliminary Work.

In the examples given below, the values of the factors are in some cases taken to several places of decimals. Where strict accuracy is not required, as in preliminary designs, interpolation may be omitted and the computations can be still further reduced by working to fewer places of decimals. For instance, at the end of Example 3, we have:—

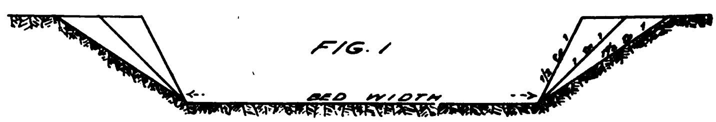
$$\begin{aligned} Q &= a\sqrt{r} \times c \times \sqrt{s} \\ &= 729.5 \times 81.4 \times .016854 \\ &= 1000 \text{ cubic feet per second.} \end{aligned}$$

Instead of taking $a\sqrt{r} = 729.5$ let us omit decimals and take it $= 729$, and in table 23 with $n = .0225$ under a slope of 1 in 3333 and opposite $\sqrt{r} = 1.9$ we find,

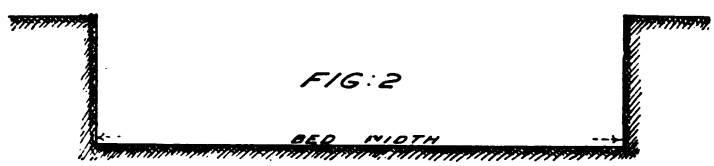
omitting decimals that $c = 82$, and for 1 in 3520 let us take $\sqrt{s} = .0168$ instead of .016854. Substituting these values in formula (45) we have:—

$$Q = 729 \times 82 \times .0168$$
$$= 1004 \text{ cubic feet per second,}$$

being in excess less than one-half of one per cent. which is near enough for preliminary work for all practical purposes.

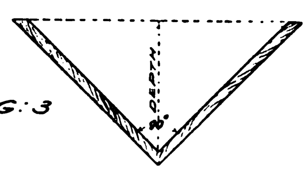


TRAPEZOIDAL CHANNELS



RECTANGULAR CHANNEL

FIG: 3



V-FLUME.

Article 13. Explanation and Use of the Tables.

After the dimensions, slope, etc., of the channel have been determined by the use of the Tables, it is advisable, in order to take every precaution to obtain accuracy, that, as a final check, the work should be computed by Kutter's formula (40).

EXAMPLE 1.—*To find mean velocity and discharge of a canal.*

Required the mean velocity and discharge of a canal having a bed width of 70 feet, a depth of water of 4 feet, with side slopes of $1\frac{1}{2}$ to 1, a longitudinal slope or grade of 1 in 1544, and with the co-efficient of the surface of the material of the bed$=.03$.

State also the quantity of land this canal will irrigate, the duty of water being 190 acres per cubic foot per second.

The velocity and discharge may be found by three mothods:

First, by arithmetic.

Second, by logarithms.

Third, by the tables in this work.

We will compute the above example by each of these methods.

1. Computing by arithmetic:—

$$s=\frac{1}{1544}=.000647668, \text{ and } \sqrt{s}=.025449.$$

In Table 33 of slopes, the value of s and $\sqrt{s}$ can be found quickly by inspection.

Area of water section$=(70+6)\times4=304$

Perimeter of water section$=70+2\times\sqrt{6^2+4^2}=84.42$

Hydraulic mean depth $r=\frac{a}{p}=\frac{304}{84.42}=3.601.$

$$\text{and } \sqrt{r}=\sqrt{3.601}=1.9$$

Kutter's formula is:—

$$v=\left\{\frac{\frac{1.811}{n}+41.6+\frac{.00281}{s}}{1+\left(41.6+\frac{.00281}{s}\right)\frac{n}{\sqrt{r}}}\right\}\times\sqrt{rs}$$

Substituting the values of n, s and r above given, in this formula, we have:—

$$v=\left\{\frac{\frac{1.811}{.03}+41.6+\frac{.00281}{.000647668}}{1+\left(41.6+\frac{.00281}{.000647668}\right)\times\frac{.03}{1.9}}\right\}\times\sqrt{3.6\times .000647668}$$

Computing this equation we find

$v=2.98$ feet per second; and

$Q=2.98\times 304=906$ cubic feet per second.

2. Computing by logarithms:—

First, we compute the value of each term in the numerator of the large parenthesis, and take their sum.

Second, we compute the value of each term in the denominator, and take their sum.

Third, find the value of numerator divided by denominator, and this is equal to c.

Fourth, find the value of $\sqrt{rs}$ and multiply it by c, and this last result is equal to v.

From log 1.811 = 0.2579
Deduct log .03 = -2.4771

1.7808 log of 60.370

The second term is 41.600

From log .00281 = -3.4487
Deduct log .0006477 = -4.8114

0.6373 log of.. 4.338

∴ Numerator in large parenthesis.....106.308

For the denominator we add the values already found of the second and third terms of the numerator:—

$$41.600+4.338=45.938=45.94 \text{ nearly,}$$
$$\text{and log of } 45.94 = 1.6622.$$

From log .03 $= -2.4771$
deduct log $3.601 \div 2 = 0.2782$
$= -2.1989$

-1.8611 log of 0.7262
Add first term in denominator...............1.000

1.7262

And $c = \dfrac{106.308}{1.7262} = 61.59.$

As $v = c\sqrt{rs}$, we have now to find value of $\sqrt{rs}$

log 3.601 $= 0.5564$
log .0006477 $= -4.8114$
$- 3.3678$

and this $\div 2 = -2.6839$, the number corresponding to which is $.0483 = \sqrt{rs}$

$\therefore v = c\sqrt{rs} = 61.59 \times .0483 = 2.975$ feet per second.
$Q = a \times v = 304 \times 2.975 = 904.4$ cubic feet per second

3 Computing by tables in this work.

Look out, in Table 11, under bed width 70 feet, and opposite depth 4 feet, and we find $a = 304$, $r = 3.601$, $\sqrt{r} = 1.9$, $a\sqrt{r} = 578$. Look out, in Table 26, where $n = .03$ and $\sqrt{r} = 1.9$, and under the slope 1 in 1666 (which is the nearest slope to 1 in 1544), and we find $c = 61.6$, and $c\sqrt{r} = 117.0$. In Table 33 the nearest slope to 1 in 1544 is 1545, and the $\sqrt{s}$ of $1545 = .025441$. Substitute the values of $c\sqrt{r}$ and $\sqrt{s}$ in formula (41),

$$v = c\sqrt{r} \times \sqrt{s}$$

and we have

$v = 117 \times .025441 = 2.98$ feet per second.
$Q = va = 2.98 \times 304 = 906$ cubic feet per second.

Now, as a check on this, we substitute the values of the other factors given, and we have

$$v=c\times\sqrt{r}\times\sqrt{s}$$

$$=61.6\times1.9\times.025441=2.99 \text{ feet per second.}$$

$$Q=c\times a\sqrt{r}\times\sqrt{s}$$

$$=61.6\times578\times.025441=906 \text{ cubic feet per second.}$$

As each cubic foot per second will irrigate 190 acres of land, we have $906\times190=172{,}140$ acres, the area which the canal can irrigate.

This example shows the great saving of time and labor effected by the use of the tables, and with the additional advantage of having a check on the accuracy of the work.

EXAMPLE 2.—*Given the discharge, bottom width and depth to find the grade of channel.*

A canal is designed to discharge 410 cubic feet per second. It is to be 30 feet on bed, 4 feet deep, with side slopes of 1 to 1. What is the grade necessary to produce the given discharge, the value of n being .025 ?

First method:

The area of section=136 square feet, and

$$v=\frac{Q}{a}=\frac{410}{136}=3 \text{ feet.}$$

Look out, in Table 8, under a bed width of 30 feet and a depth of 4 feet, and we find $\sqrt{r}=1.81$.

In Table 32, with a value of $n=.03$, the slope of 1 in 1323 is found in a channel of the given dimensions to produce a velocity of 3 feet per second.

Now, in Table 24, with $n=.025$, and under a slope of 1 in 1250, which is the nearest slope to 1323, and opposite $\sqrt{r}=1.8$, we have $c=72.3$; and similarly, in Table 26, with $n=.03$ we have $c_1=60.2$.

But $l : l_1 :: c_1^2 : c^2$ substitute values and

$$l_1 = \frac{1323 \times 72.3^2}{60.2^2} = 1908$$

Therefore the approximate slope is 1 in 1908.

Second method:

Finding by Table 32, as a first approximation, that when $n = .03$ the slope to produce the given velocity is 1 in 1323, and we therefore know that when $n = .025$ the slope must be flatter to give the same velocity. We therefore look out in the next flatter slope, in Table 24, with $n = .025$, which we find is 1 in 1666, and we find opposite $\sqrt{r} = 1.81$ that $c\sqrt{r} = 131.11$.

Substituting the value of $c\sqrt{r}$ and v in formula (43),

$$\sqrt{s} = \frac{v}{c\sqrt{r}}$$

and we have

$$\sqrt{s} = \frac{3}{131.11} = .022883$$

$$\therefore \sqrt{s} = .022883$$

$$s = .022883^2$$

$$\text{and } \frac{1}{s} = \frac{1}{.022883^2}$$

But $\frac{1}{s}$ = ratio of slope.

Now by logarithms:

From log 1 .... 0.0000000

deduct log .022883 = $\bar{2}.3595130 \times 2$ = ... $\bar{4}.7190260$

3.2809740

which corresponds to 1910. As the computed slope 1 in 1910 has almost the same value of c as the assumed slope 1 in 1666, therefore the required slope is 1 in 1910.

Third method:

Find in the same way as shown in Second Method, that

$$\sqrt{s}=.022883$$

Now, in Table 33 of slopes, look out the slope corresponding to this $\sqrt{s}$, and it will be found equal to 1 in 1910. This is the quickest method of finding the slope.

As a check on this work: by formula (45),

$$Q=a\times c\sqrt{r}\times\sqrt{s}$$

Substitute the values of a, $c\sqrt{r}$ and $\sqrt{s}$, and we have:

$$Q=136\times 131.1\times .022883$$
$$=408 \text{ cubic feet per second.}$$

EXAMPLE 3.—*Given the discharge, bottom width and grade of canal, to find the depth.*

A main irrigation canal has a bed width of 100 feet, side slopes of 1 to 1, and an inclination of 18 inches per mile. At what depth, above the bed of the main canal, must the sill of the head gate of a branch canal be placed, so that the main canal will be flowing 1,000 cubic feet per second, before any water flows into the branch canal; $n=.0225$? By Table 33, 18 inches per mile=1 in 3520, and $\sqrt{s}=.016854$.

By formula (47),

$$ac\sqrt{r}=\frac{Q}{\sqrt{s}}=\frac{1000}{.016854}=59333$$

Therefore, the product of the factors $a\sqrt{r}$ and $c=59333$.

All that is now required is to find in Table 8, and under a bed width 100 feet, such a depth that the product of the factors c ($n=.0225$) and $a\sqrt{r}$ shall be$=59333$.

In Table 8, and under bed width 100 feet, look down column $a\sqrt{r}$, and also in Table 23, under slope 1 in 3333 (which is the nearest slope to 1 in 3520), look down col-

umn c, and opposite the same or nearly the same value of $\sqrt{r}$ in each column, until the product of the two factors is equal or nearly equal to 59333.

Thus, in Table 8, under a bed width of 100 feet and depth 3.75, we find $\sqrt{r}=1.875$ and $a\sqrt{r}=729.5$. In Table 23, under a slope of 1 in 3333, we find opposite

$$\sqrt{r}=1.8 \quad \text{that } c=80.2$$
$$\therefore \sqrt{r}=0.075 \text{ that } c=1.2$$
$$\therefore \sqrt{}=1.875 \text{ that } c=81.4$$

Now, $729.5 \times 81.4=59381$, being near enough, for all practical purposes, to 59333.

The required depth is, therefore, 3.75 feet.

As a check on the above, let us compute the discharge of the channel with the depth found. In Table 8, under a bed width of 100 feet, and depth of 3.75 feet, the value of $a\sqrt{r}=729.5$. In Table 23, under a slope of 1 in 3333, we find by interpolation that when $\sqrt{r}=1.875$ that $c=81.4$.

As before found, for 18 inches per mile $\sqrt{s}=.016854$.

Now substitute these values of $a\sqrt{r}$, c and $\sqrt{s}$, in formula (45):—

$$Q=a\sqrt{r}\times c\times\sqrt{s} \text{ and we have}$$
$$Q=729.5\times 81.4\times .016854$$
$$=1000 \text{ cubic feet per second.}$$

EXAMPLE 4.—*Given the hydraulic mean depth and mean velocity of a channel, to find the slope or grade.*

A canal has a hydraulic mean depth of 9.18 feet, and a mean velocity of 5.5 feet per second. The canal is trapezoidal in cross-section, but slightly rounded, and it is free from detritus. Under these favorable conditions

the value of n is assumed $=.0225$. What is the slope of the water surface of this canal?

r being $= 9.18 \therefore \sqrt{r} = 3.03$.

We assume as an approximation that the slope is 1 in 1800.

We look out, in Table 23 ($n=.0225$), and under slope 1 in 1666.6, which is nearest to 1 in 1800, and opposite

$\sqrt{r} = 3.03$, we find the value of $r = 94.3$, and

$\therefore c\sqrt{r} = 94.3 \times 3.03 = 285.7$.

Now, formula (43), $\sqrt{s} = \frac{v}{c\sqrt{r}} = \frac{5.5}{285.7} = .019251$.

Now look out, in Table 33, and the nearest value of $\sqrt{s}$ to .019251 is .019245 opposite a slope of 1 in 2700.

We thus find a slope of 1 in 2700, but as the assumed slope was 1 in 1800, we will compute again for a closer approximation with the slope 1 in 2700.

In Table 23, and opposite $\sqrt{r}=3.03$, and between slopes 1 in 2500 and 1 in 3333, we interpolate, and find the value of c to be $= 95$, and $c \times \sqrt{r} = 95 \times 3.03 = 287.85$.

Now, formula (43), $\sqrt{s} = \frac{v}{c\sqrt{r}} = \frac{5.5}{287.85} = .019107$.

Look out, in Table 33 of slopes, and opposite $\sqrt{s}=$ .019104, the nearest one in the table to .019107, we find the slope to be 1 in 2740, which is the slope required.

EXAMPLE 5.—*Given the discharge, velocity and grade of a channel, to find the bed width and depth.*

What must be the bed width and depth of a canal to discharge 300 cubic feet per second, at a mean velocity of 2 feet per second? The side slopes are $1\frac{1}{2}$ to 1; inclination, 16 inches to the mile; and $n=.025$.

In Table 33, we find 16 inches in a mile = a slope of 1 in 3960, and also $\sqrt{s}=.015891$.

By formula (46), $a=\frac{Q}{v}=\frac{300}{2}=150$ square feet.

By formula (42), $c\sqrt{r}=\frac{v}{\sqrt{s}}=\frac{2}{.015891}=125.9$.

Look out now, in Table 24, with $n=.025$, and under slope 1 in 3333 (which is nearest to 1 in 3960), and we get

$$c\sqrt{r}=119.9, \text{ value of } \sqrt{r}=1.7$$
$$c\sqrt{r}=130.1, \text{ value of } \sqrt{r}=1.8$$

Therefore for $c\sqrt{r}=125.9$, value of $\sqrt{r}$ may be taken at 1.75.

$$\therefore a \times \sqrt{r}=150 \times 1.75=262.5$$

Look now in Table 11, and under the same bed width for a value of $\sqrt{r}=1.75$, and $a\sqrt{r}=262.5$, and we find the nearest values of these factors to be, under a bed width of 35 feet and depth 3.5 feet, $\sqrt{r}=1.72$, and $a\sqrt{r}=242.4$; and, depth 3.75 feet, $\sqrt{r}=1.77$, and $a\sqrt{r}=269.6$.

$$\text{Now } 262.5-242.4=20.1,$$
$$\text{and } 269.6-242.4=27.2.$$
$$\therefore 27.2 : \text{depth } .025 :: 20.1 : .19,$$

the required increased depth, approximately, over 3.5 feet is 0.19 feet. The approximate depth is therefore $3.5+0.19=3.69$ feet.

As a check, we will now find the discharge with this depth, 3.69 feet, and a bed width of 35 feet. The area of section = 149.57 square feet. The perimeter = 48.01.

The value of $r=\frac{a}{p}=\frac{149.57}{48.01}=3.1154$.

And $\sqrt{r}=1.77$.

In Table 24, with $n=.025$, and under a slope of 1 in 3333, we find, when

$$\sqrt{r}=1.7\ \text{, that } c=70.5$$

and by interpolation, $\sqrt{r}=0.07$, that $c=\ 1.3$

and $\therefore$ $\sqrt{r}=1.77$, that $c=\overline{71.8}$

Now substitute values of c, $\sqrt{r}$, and $\sqrt{s}$, in formula (41)

$$v=c\times\sqrt{r}\times\sqrt{s},$$

and we have:

$v=71.8\times1.77\times.015891=2.0195$ feet per second;
and $Q=av=149.57\times2.0195=302$ cubic feet per second.

This is near enough for most purposes, but if the exact dimensions be required, one square foot can be taken off the area by diminishing either the depth or bed width of channel, and as the velocity is 2 feet per second, the discharge will then be 300 cubic feet per second.

EXAMPLE 6.—*Gauging a stream to find its velocity and discharge, and the number of acres it is capable of irrigating.*

It is required to ascertain how many acres of orchard land a stream will irrigate, the duty of the water being assumed at 400 acres per cubic foot per second.

In a straight reach of the stream, and where it was tolerably uniform, three cross-sections were taken 300 feet apart.

The first had an area $=22.3$ square feet, and wetted perimeter $=14.76$ lineal feet.

The second had an area $=23.1$ square feet, and wetted perimeter $=14.07$ lineal feet.

The third had an area $=23.9$ square feet, and wetted perimeter $=13.68$ lineal feet.

The surface slope of the stream was found, by level-

ing, to fall 0.287 feet in 600 feet. As the stream was irregular, and was choked occasionally with vegetation, the value of n was assumed at .03. We have now the information required to find the discharge of the stream.

Add the three areas, and divide by 3, and we get the mean area = 23.1 square feet. Again add the three wetted perimeters and divide by 3, and we find the mean perimeter = 14.17 lineal feet.

$$\text{Now } r=\frac{a}{p}=\frac{23.1}{14.17}=1.63,$$

$$\text{and } \sqrt{1.63}=1.28 \text{ feet.}$$

A slope of 0.287 feet in 600 feet = 1 in 2090, and Table 33 for this slope, $\sqrt{s}=.021874$.

In Table 26, with $n=.03$, we do not find a slope of 1 in 2090.

We do, however, for 1 in 1666, and $\sqrt{r}=1.2$, that $c=49.4$;
and for 1 in 2500, and $\sqrt{r}=1.2$, that $c=49.2$;
and as 1 in 2090 is about a mean of these slopes, we take

$$\text{for } \sqrt{r}=1.2, \quad \text{that } c=49.3$$

$$\text{and for } \sqrt{r}=0.08, \text{ that } c=\ \ 1.76$$

$$\text{therefore for } \sqrt{r}=1.28, \text{ that } c=51.06$$

Substituting the values of c, $\sqrt{r}$, and $\sqrt{s}$, in formula (41),

$v=c\sqrt{r}\times\sqrt{s}$, we have

$v=51.06\times1.28\times.021874=1.43$ feet per second.

$Q=va=1.43\times23.1=33.033$ cubic feet per second.

But as each cubic foot per second is capable of irrigating 400 acres, we have $33.033\times400=13{,}213$ acres, the quantity of land the stream is capable of irrigating.

EXAMPLE 7.—*Given the dimensions of a canal in earth, to find the width of a masonry channel having the same discharge, the two channels having the same depth and grade.*

A canal in earth, with $n = .0275$, a bed of 50 feet, depth 4 feet, side slopes of ½ to 1, and a grade of 2 feet per mile, is passed over a river by a masonry aqueduct. The aqueduct is to be rectangular in cross-section, with the same depth as the canal, and the same grade. What must be the width of the masonry channel to discharge the same quantity of water as the canal, its value of n being taken [illegible] .017? This value of n is taken high, .017, as the bed and sides of the aqueduct are to be roughly plastered.

For earthen channel, Q [illegible]

Substitute the values of the factors at the right-hand side of the equation, and we have

$$Q = 66.9 \times 391 \times [illegible]$$

[illegible] cubic feet per second.

[illegible]

21, n=.017, that under a slope of 1 in 2500, and opposite $\sqrt{r}$=1.8, the value of c=106.2, and we have 248.9 × 106.2=26433, which is near enough to 26152; therefore the required width is 35 feet.

As a check on this work, look out, in Table 13, the value $a\sqrt{r}$ for a rectangular channel 35 feet wide and 4 feet deep, and substitute this, and also the value of c and $\sqrt{s}$, in formula (45),

$$\begin{aligned} Q &= c \times a\sqrt{r} \times \sqrt{s} \\ &= 106.2 \times 248.9 \times .019463 \\ &= 514 \text{ cubic feet per second,} \end{aligned}$$

which is near enough for all practical purposes.

EXAMPLE 8.—*Increased discharge of an earthen channel by clearing it of grass and weeds.*

A drainage channel originally excavated to a bed width of 12 feet, a depth of water of 4 feet, with side slopes of 1 to 1, and a grade of 1 in 1760, or 3 feet per mile, has been for some years neglected, and its bed and banks are covered with long grass and weeds. Assuming its value of n in this state = .035, what will be its increase in discharge when it is cleared of all grass, weeds and sharp bends? In the latter case we will assume n=.025.

Let us first find the discharge in the obstructed channel.

In Table 8 we find a=64, and $\sqrt{r}$ = 1.657.

In Table 33 we find, opposite a slope of 1 in 1760, that $\sqrt{s}$ = 023837.

In Table 27, with n=.035 under slope of 1 in 1666.7 (which is the nearest to 1 in 1760), and opposite $\sqrt{r}$ = 1.657, the value of c =49.55.

Now substitute the values of a, c, $\sqrt{r}$ and $\sqrt{s}$, in formula (45),

$$Q = ac\sqrt{r} \times \sqrt{s}$$
$$= 64 \times 49.55 \times 1.657 \times .023837$$
$$= 125.3 \text{ cubic feet per second,}$$

being the discharge of the obstructed channel.

Let us now find the discharge of the same channel after it has been cleared, ***at slight expense,*** of brush, weeds, silt deposit, sharp bends, etc., so as to bring its value of $n = .025$.

In Table 24, with $n = .025$, under a slope of 1 in 1666.7 and opposite $\sqrt{r} = 1.657$ (found by interpolation), the value of $c = 69.87$. Now substitute this value of c with the given values of a, $\sqrt{r}$ and $\sqrt{s}$ in formula (45), and we have:—

$$Q = 64 \times 69.87 \times 1.657 \times .023837$$
$$= 176.6 \text{ cubic feet per second,}$$

being the discharge of the improved channel.

We thus see that by clearing out the channel its discharge has been increased by more than 40 per cent.

EXAMPLE 9.—*Increase of discharge by improving in smoothness the masonry surface of a channel.*

A semi-circular open channel of coarse rubble set dry, of 2 feet radius and a grade of 1 in 500, and with $n = .02$, is to be improved by filling up all interstices, and giving its surface a coat of medium smooth plaster, so as to make its value of $n = .013$. What is the percentage of increase in discharge of the improved channel?

The hydraulic mean depth, r, of a circular channel flowing full or half full is equal to half the radius, therefore r of this channel = 1, and $\sqrt{r} = 1$.

The value of c for all slopes greater than 1 in 1000 is the same as for 1 in 1000.

In Table 22, with $n=.02$, under a slope of 1 in 1000 and opposite $\sqrt{r}=1$, the value of $c=71.5$.

In Table 33, opposite a slope of 1 in 500, the value of $\sqrt{s}=.044721$.

Substitute the values of c, $\sqrt{r}$ and $\sqrt{s}$ in formula (41),

$$v=c\times\sqrt{r}\times\sqrt{s}$$
$$\text{and } v=71.5\times1\times.044721$$
$$v=3.2 \text{ feet per second,}$$

the velocity of the channel with a surface of coarse rubble.

Now, to find the velocity in plastered channel. Look out, in Table 19, $n=.013$, and under a slope of 1 in 1000, and opposite $\sqrt{r}=1$, we find $c\sqrt{r}=116.5$.

Substitute the values of $c\sqrt{r}$ and $\sqrt{s}$, and we have

$$v=116.5\times.044721$$
$$=5.2 \text{ feet per second,}$$

the mean velocity in the plastered channel; which shows an increase in velocity and discharge of 63 per cent. over the coarse rubble channel.

EXAMPLE 10.—*To find the velocity and discharge of a channel having bed width, depth and side slopes not given in the tables.*

What is the velocity and discharge of a channel having bed width 110 feet, depth of water 7.2 feet, side slopes 2 to 1, and grade 1 in 5000, the value of n being equal to .0275?

$$a=110+(7.2\times2)\times7.2=895.68 \text{ square feet.}$$

In Table 29 of length of side slope, we find, under a slope of 2 to 1 and opposite 1 foot, 4.472 feet. Multiply this by the depth, 7.2, and we have the length of two side slopes, and, therefore:—

$$p = 110 + (4.472 \times 7.2) = 142.2$$

$$r = \frac{895.68}{142\ 2} = 6.3$$

$$\text{and } \sqrt{r} = \sqrt{6.3} = 2.51.$$

In Table 25, with $n = .0275$, under a slope of 1 in 5000, and opposite $\sqrt{r} = 2.51$, the value of $c = 75.5$.

In Table 33, opposite a slope of 1 in 5000, the value of $\sqrt{s} = .014142$.

Substitute the values of c, $\sqrt{r}$ and $\sqrt{s}$ in formula (41),

$$v = c \times \sqrt{r} \times \sqrt{s}$$

$$\text{and } v = 75.5 \times 2.51 \times .014142$$

$$= 2.68 \text{ feet per second,}$$

$$\text{and } Q = a v$$

$$= 895.68 \times 2.68$$

$$= 2400 \text{ cubic feet per second.}$$

EXAMPLE 11.—*Given the discharge, grade and ratio of bed width to depth, to find bed width and depth.*

A mining ditch is to discharge 130 feet per second, and its grade is 1 in 1000. What must be its bed width and depth the ratio of bed width to depth being as 2 to 1? Its side slopes are to be $\frac{1}{2}$ to 1, and its value of $n = .025$.

By Table 33, a slope of 1 in 1000 has $\sqrt{s} = .031623$.

Substitute the value of s and also the value of Q given in formula (47),

$$a c \sqrt{r} = \frac{Q}{\sqrt{s}} = \frac{130}{.031623} = 4111.$$

Now look out the value of the factors c and $a\sqrt{r}$, in Tables 10 and 24, until their product is equal or nearly equal to 4111. The value of c is found in Table 24 with $n=.025$, under the given slope 1 in 1000, and opposite the $\sqrt{r}$ corresponding to the value of $a\sqrt{r}$.

After inspection, we find in Table 10, under a bed width 8 feet and depth 4 feet, that $a\sqrt{r}=61.47$, and $\sqrt{r}=1.54$.

Also in Table 24, with $n=.025$, under a slope of 1 in 1000 and opposite $\sqrt{r}=1.54$, we find $c=67.9$; therefore, $ac\sqrt{r}=61.47\times67.9=4131$, which is sufficiently near to 4111 for practical work.

Let us check this discharge.

$$\begin{aligned} Q&=c\times a\sqrt{r}\times\sqrt{s} \\ &=67.9\times61.47\times.031623 \\ &=132 \text{ cubic feet per second.} \end{aligned}$$

The dimensions of the channel are therefore 8 feet wide on bed, 4 feet deep, and with side slopes of ½ to 1.

EXAMPLE 12.—*Diminution of discharge of channel by grass and weeds.*

The above channel, Example 11, after construction, has not been repaired or cleaned out for several years. It is obstructed by grass and weeds, and its value of n increased to .035. Find the percentage of diminution of discharge.

In Table 27, with $n=.035$, under a slope of 1 in 1000 and opposite $\sqrt{r}=1.54$, we find $c=47.8$.

Substituting this value, and also the values $a\sqrt{r}$ and $\sqrt{s}$, in formula (45), we have:—

$$\begin{aligned} Q&=c\times a\sqrt{r}\times\sqrt{s} \\ &=47.8\times61.47\times.031623 \\ &=92.9 \text{ cubic feet per second.} \end{aligned}$$

This shows that, in this case, the grass and weeds diminished the discharge by about 30 per cent. of the original discharge.

EXAMPLE 13.—*Given discharge, velocity and the ratio of bed width to depth, to find the slope or grade.*

A canal is to discharge 3000 cubic feet per second. Its mean velocity is to be 2.5 feet per second. Its bed width is to be 15 times the depth, its side slopes 1 to 1, and its value of $n = .025$. Find the slope required.

$$a = \frac{Q}{v} = \frac{3000}{2.5} = 1200 \text{ square feet.}$$

Let x = depth; then

$$x \times 16x = 16x^2 = 1200$$

$$\therefore x = 8.66$$

$$p = (8.66 \times 15) + (8.66 \times 2.828) = 154.39$$

$$r = \frac{a}{p} = \frac{1200}{154.39} = 7.772.$$

$$\sqrt{r} = \sqrt{7.772} = 2.8 \text{ nearly.}$$

In order to aid in the selection of the slope, look out in Table 32, with $n = .03$, under bed width 140 feet, depth 9 feet, and we find, as a rough approximation, that the slope for a velocity of $2\frac{1}{2}$ feet per second is $= \dfrac{11453 + 4822}{2} = 8138$, that is, 1 in 8138. But as the slope for $n = .025$ is flatter than when $n = .03$, we may assume a flatter slope than 1 in 8138. The nearest slope to this in the tables is 1 in 10000.

We now find in Table 24, with $n = .025$, under a slope of 1 in 10000, and opposite $\sqrt{r} = 2.8$, that $c\sqrt{r} = 245.3$.

Now substitute the values of $c\sqrt{r}$ and v in formula (43),

$$\sqrt{s}=\frac{v}{c\sqrt{r}}$$

and we have $\sqrt{s}=\dfrac{2.5}{245.3}=.010191.$

Now look out in Table 33, and the nearest value of $\sqrt{s}$ to this will be found opposite a slope of 1 in 9600.

As a check on this, find value of c in Table 24, under a slope of 1 in 10000, and opposite $\sqrt{r}=2.8$, we find it equal to 87.6.

$$\begin{aligned}\therefore\ v&=c\times\sqrt{r}\times\sqrt{s}\\&=87.6\times 2.8\times .010191\\&=2.5 \text{ feet per second}\\Q&=av=1200\times 2.5\\&=3000 \text{ cubic feet per second.}\end{aligned}$$

EXAMPLE 14.—*Given the bed width, depth and grade of a channel not given in the tables, to find the velocity and discharge.*

A canal has a bed width of 80 feet, a depth of six feet, and side slopes of 1½ to 1. Its grade is 1 in 5000, and its value of $n=.025$. Find its velocity and discharge.

The table for channels with side slopes of 1½ to 1 does not extend beyond a bed width of 60 feet; but, as before explained, the velocity in channels having a greater bed width than 60 feet is not practically changed by a change in the side slopes usually adopted; that is, as an instance, the velocity in a channel 80 feet wide and 6 feet deep, with side slopes of 1 to 1, is practically the same as a channel having the same width and depth but with side slopes of 1½ to 1.

Let us, therefore, find first the velocity in the former channel.

In Table 8, with side slopes of 1 to 1, under a bed width of 80 feet, and opposite a depth of 6 feet, the value of $\sqrt{r}=2.307$.

In Table 24, with $n=.025$, and under a slope of 1 in 5000, we find, corresponding to a value of $\sqrt{r}=2.307$, that the value of $c=80.7$.

In Table 33 of slopes, and opposite a slope of 1 in 5000, the $\sqrt{s}=.014142$.

Substitute the values of c, $\sqrt{r}$ and $\sqrt{s}$ in formula (41),

$$v=c\times\sqrt{r}\times\sqrt{s}$$

and we have $v=80.7\times 2.307\times .014142$

$$=2.63 \text{ feet per second.}$$

$$Q=av=534\times 2.63$$

$$=1404 \text{ cubic feet per second.}$$

Let us now check this.

The area of a channel 80 feet on bed, 6 feet deep, and with side slopes of $1\frac{1}{2}$ to 1, is equal to

$$(80+6\times 1\ 5)\times 6=534 \text{ square feet.}$$

In Table 29 of length of side slopes, we find opposite a depth of 6 feet, and under a slope of $1\frac{1}{2}$ to 1, that the length of the two side slopes $=21.634$ feet. To this has to be added bed width 80 feet, making the perimeter $=$ 101.634 feet.

$$\text{Now } r=\frac{a}{p}=\frac{534}{101.634}=5.2541$$

and $\sqrt{r}=2.292$.

We have already found that the value of $\sqrt{r}$ with side slopes of 1 to 1 is 2.307, showing a difference of less than 1 per cent. with side slopes of $1\frac{1}{2}$ to 1.

We therefore see that, for all practical purposes, the velocity found from the tables with side slopes of 1 to 1 is sufficiently correct.

EXAMPLE 15.—*To find the value of c and n in an open channel.*

A channel is gauged, and its perimeter is found equal to 26.48 lineal feet, and its area equal to 63 square feet. Its discharge is 101.5 cubic feet per second, and the slope of its water surface is equal to 22 inches per mile. Find the value of c and n.

$$r=\frac{a}{p}=\frac{63}{26.48}=2.4$$

and $\sqrt{r}=\sqrt{2.4}=1.55$

$$v=\frac{Q}{a}=\frac{101.5}{63}=1.61 \text{ feet per second.}$$

In Table 33, and opposite 22 inches per mile, $\sqrt{s}=$ .018634.

Substituting the value of $\sqrt{s}$, v and s in formula,

$$c=\frac{v}{\sqrt{r}\times\sqrt{s}} \text{ we have}$$

$$c=\frac{1.61}{1.55\times .018634}=55.8$$

A slope of 1 in 2500 is the nearest in the tables of n to 22 inches per mile. Now look under the different values of n, and under a slope of 1 in 2500, and opposite $\sqrt{r}=1.55$, and the value of c that is nearest to 55.8 will be found under the required value of n. In this case, in Table 26, under a value of $n=.03$, and under a slope of 1 in 2500 and opposite $\sqrt{r}=1.5$, we find the value of $c=55.2$, which is the nearest value in the tables to 55.8. Therefore, the required value of $c=$ 55.8, and $n=.03$.

As a check on this, look out in Table 26, with $n=.03$, under a slope of 1 in 2500, and opposite $\sqrt{r}=1.55$, and

c is found, by interpolation, $=56.1$. Substitute this value of c, and also the values of $\sqrt{r}$ and $\sqrt{s}$, in formula (41),

$$v=c\times\sqrt{r}\times\sqrt{s}$$

and we have $v=56.1\times1.55\times.018634$
$=1.62$ feet per second.

EXAMPLE 16.—*To find the velocity and discharge of a brick aqueduct by Bazin's formula, the dimensions and grade being given.*

An aqueduct constructed of brick work, rectangular in cross-section, 4 feet wide on bottom, and with vertical sides, carries 2 feet in depth of water and has a slope of 1 in 160. What is its velocity and discharge by Bazin's formula for open channels?

In Table 13 for rectangular channels, we find under a bed width 4 and opposite depth 2 that $\sqrt{r}=1$. As the channel is of brick-work, it comes under the head of the second type of Bazin's channels, formula (35), by which Table 28 is computed. Now, in Table 28, and opposite $\sqrt{r}=1$, we find that $c\sqrt{r}=118.5$.

We also find, in Table 33, and opposite a slope of 1 in 160, that $\sqrt{s}=.079057$. Substituting this value and also the value of $c\sqrt{r}$ in formula (41),

$$v=c\sqrt{r}\times\sqrt{s}$$

we have $v=118.5\times.079057=9.37$ feet per second
and $Q=a\,v=8\times9.37=74.96$ cubic feet per second.

EXAMPLE 17.—*Increase of discharge of a channel in rock-cutting by plastering its surface.*

Near the head of a small irrigation canal the supply of water is carried in a rock-cutting 10 feet wide at bottom, 12 feet wide at surface of water 5 feet in depth, and having a slope of 1 in 880.

The water supply carried in this cutting being insufficient, it is determined to increase the supply without, however, increasing the cross-sectional area of channel or its slope. The bottom and sides of the rock-cutting are very rough, and in order to give them a smoother surface and increase the discharge, it is determined to fill up all the hollows in them with masonry, and after this to lay on carefully a coat of cement plaster with one-third sand, and to make the surfaces in contact with the water smooth and even.

After the plastering is finished the dimensions of the channel will be: width at bottom 9.8 feet, width at water surface 11.8 feet, depth of water 4.9 feet, and the slope as before, 1 in 880.

It is assumed that a near approximation to the value of n for the rock-cutting $=.0225$, and for the plastered channel $n=.011$.

Find the increase in discharge in the plastered channel over that in the original channel.

In the original channel

$$r=\frac{\text{area}}{\text{wetted perimeter}}=\frac{55}{20.2}=2.7228$$

$$\sqrt{r}=\sqrt{2.7228}=1.65$$

Table 33 shows, for a slope of 1 in 880, that $\sqrt{s}=.03371$.

Table 23 shows, by interpolation, under a slope of 1 in 1000 (which has the same co-efficient as a slope of 1 in 880), and opposite $\sqrt{r}=1.65$, that $c\sqrt{r}=128.4$.

Substitute the values of $\sqrt{s}$ and $c\sqrt{r}$, in formula (41), and we have:—

$$v=128.4\times .03371=4.328 \text{ feet per second.}$$

Now, $Q=va=4.328\times 55=238$ cubic feet per second.

In the plastered channel $r=\frac{a}{p}=\frac{52.92}{19.8}=2.673$ and $\sqrt{r}=\sqrt{2.673}=1.64$ nearly.

In Table 17, with $n=.011$, we find, by interpolation, under a slope of 1 in 1000 and opposite $\sqrt{r}=1.64$, that the value of $c\sqrt{r}=264.2$.

Substituting this value of $c\sqrt{r}$ and $\sqrt{s}$, in formula (41), and we have:—

$$v=264.2\times.03371=8.9 \text{ feet per second,}$$

and $Q=v\,a=8.9\times 52.92=471$ cubic feet per second.

We here see the effect of a smooth surface in increasing the velocity and discharge of a channel. Although the cross-sectional area has been *diminished*, still the effect of giving a smooth surface to the channel has been to *more than double its velocity* and to *almost double the discharge*. The old formula would give almost the same velocity and discharge to the two channels, as these formulæ do not take into account the surfaces exposed to the flow of water.

FLUMES.

EXAMPLE 18.—*To find the velocity and discharge of a rectangular flume.*

A rectangular flume 8 feet wide, and flowing 4 feet in depth of water, has a slope of 1 in 500. The flume is old, and its surface exposed to the flow of water is rough. Its value of n is, therefore, taken as $=.015$. Find its velocity and discharge.

In Table 13, for rectangular channels, under a bed of 8 feet, and opposite a depth of 4 feet, we find $\sqrt{r}=1.414$.

As the value of c for all slopes steeper than 1 in 1000 is the same as for 1 in 1000, we now find in Table 20,

with $n = .015$, under a slope of 1 in 1000, and with $\sqrt{r} = 1.414$, that the value of c by interpolation $= 112.25$.

In Table 33 of slopes, we find that 1 in 500 has a value $\sqrt{s} = .044721$.

Substitute these three values in formula (41),

$$v = c \times \sqrt{r} \times \sqrt{s}$$

and we have $v = 112.25 \times 1.414 \times .044721$

$$= 7.1 \text{ feet per second.}$$

$$Q = a\,v = 32 \times 7.1 = 227.2 \text{ cubic feet per second.}$$

EXAMPLE 19.—*To find the velocity and discharge of a V-flume.*

A right-angled V-flume is flowing with a depth of water in the center of 9 inches and grade of 1 in 180. Find its velocity and discharge.

The flume is new and made of unplaned timber, and its surface exposed to the water continuous on the inside, and in fairly good condition. Its value of n may therefore be taken $= .012$, but, to be on the side of safety, it is taken $= .013$.

In Table 14, for V-flumes with $n = .013$, and opposite a depth of .75 feet, we find $a = .56$ square feet, $c\sqrt{r} = 44.55$, and $a c\sqrt{r} = 24.95$.

In Table 33 of slopes, we find for a slope of 1 in 180 that $\sqrt{s} = .074536$.

Substitute the values of $c\sqrt{r}$ and $\sqrt{s}$ in formula (41),

$$v = c\sqrt{r} \times \sqrt{s}$$

and we have $v = 44.55 \times .074536$

$$= 3.32 \text{ feet per second;}$$

and $Q = a\,v = .56 \times 3.32$

$$= 1.86 \text{ cubic feet per second.}$$

As a check on this we have formula (45),

$$[illegible]$$

Substitute values, and

$$Q = 24.[illegible]\ .074[illegible]$$
$$= 1.[illegible]6 \text{ cubic feet per second.}$$

EXAMPLE 2[illegible].—*Given [illegible] width, depth and discharge of a [illegible], to find its grade or slope.*

Find, by Kutter's formula, the slope of a flume constructed of unplaned planks, 5 feet wide at bottom, with vertical sides 2½ feet high, in order that it may discharge 102 cubic feet per second.

In Table 13 under a bed width of 5 feet and opposite a depth of 2.5 feet, we find $\sqrt{r}$ = 1.118 = 1.12, nearly.

Let us assume that Table 18, with $n = .012$, is applicable to this channel and in it, under a slope of 1 in 1000, we find

$$[illegible]\ 1.1\ \text{that}\ [illegible]$$
$$[illegible]\ 1.2\ \text{that}\ [illegible].7$$
$$\therefore\ [illegible]\ 1.12\ \text{that}\ = 132.3$$

$$[illegible] = \frac{102}{12.5} = 8.16 \text{ feet per second.}$$

Substitute the values [illegible] in formula (43),

$$[illegible] \text{ and we have}$$

$$\frac{8.16}{132.3 \times 1.12} = .05507.$$

Now look out, in Table 3[illegible], the nearest value of $\sqrt{s}$ to this, and we find it to be opposite a slope of 1 in 330, which is the slope required.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 1 Foot.				Bed 2 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	0.75	0.311	0.577	.433	1.25	0.366	.605	.756	0.5
0.75	1.31	0.425	0.652	.856	2.06	0.500	.707	1.46	0.75
1.	2.	0.522	0.723	1.45	3.	0.621	.788	2.36	1.
1.25	2.81	0.620	0.787	2.21	4.06	0.734	.856	3.48	1.25
1.5	3.75	0.715	0.846	3.17	5.25	0.841	.917	4.8	1.5
1.75	4.81	0.809	0.899	4.32	6.56	0.942	.971	6.4	1.75
2.	6.	0.901	0.950	5.70	8.	1.045	1.022	8.2	2.
2.25					9.56	1.143	1.069	10.2	2.25
2.5					11.25	1.240	1.113	12.5	2.5
2.75					13.06	1.336	1.156	15.1	2.75
3.					15.	1.431	1.196	17.9	3.
3.25					17.06	1.525	1.235	21.1	3.25
3.5					19.25	1.618	1.272	24.5	3.5
3.75					21.56	1.703	1.305	28.1	3.75
4.					24.	1.803	1.342	32.2	4.

	Bed 3 Feet.				Bed 4 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	1.75	0.396	0.629	1.1	2.25	0.416	0.645	1.5	0.5
0.75	2.81	0.549	0.741	2.1	3.56	0.582	0.763	2.7	0.75
1.	4.	0.686	0.828	3.3	5.	0.732	0.856	4.3	1.
1.25	5.31	0.812	0.901	4.8	6.56	0.871	0.933	6.1	1.25
1.5	6.75	0.932	0.965	6.5	8.25	1.000	1.000	8.3	1.5
1.75	8.31	1.045	1.022	8.5	10.06	1.124	1.060	10.7	1.75
2.	10.	1.155	1.075	10.8	12.	1.243	1.115	13.4	2.
2.25	11.81	1.261	1.123	13.3	14.06	1.357	1.165	16.4	2.25
2.5	13.75	1.365	1.168	16.1	16.25	1.468	1.211	19.7	2.5
2.75	15.81	1.466	1.211	19.1	18.56	1.576	1.255	23.3	2.75
3.	18.	1.567	1.252	22.5	21.	1.682	1.297	27.2	3.
3.25	20.31	1.666	1.290	26.2	23.56	1.786	1.339	31.5	3.25
3.5	22.75	1.764	1.328	30.2	26.25	1.889	1.375	36.1	3.5
3.75	25.31	1.851	1.364	34.5	29.06	1.990	1.411	41.0	3.75
4.	28.	1.956	1.398	39.1	32.	2.090	1.446	46.3	4.
4.25	30.81	2.051	1.432	44.1	35.06	2.189	1.480	51.9	4.25
4.5	33.75	2.146	1.465	49.4	38.25	2.287	1.512	57.8	4.5
4.75	36.81	2.240	1.497	55.1	41.56	2.384	1.544	64.2	4.75
5.	40.	2.333	1.527	61.1	45.	2.480	1.575	70.9	5.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. **Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ**

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 5 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 6 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	2.75	0.429	0.655	1.8	3.25	0.438	0.662	2.15	0.5
0.75	4.31	0.607	0.779	3.4	5.06	0.623	0.781	3.95	0.75
1.	6.	0.766	0.875	5.2	7.	0.793	0.891	6.2	1.
1.25	7.81	0.915	0.956	7.5	9.06	0.950	0.975	8.8	1.25
1.5	9.75	1.054	1.027	10.	11.25	1.098	1.048	11.8	1.5
1.75	11.81	1.186	1.089	12.9	13.56	1.238	1.113	15.1	1.75
2.	14.	1.314	1.147	16.1	16.	1.373	1.172	18.8	2.
2.25	16.31	1.436	1.198	19.5	18.56	1.502	1.226	22.8	2.25
2.5	18.75	1.553	1.246	23.4	21.25	1.626	1.275	27.1	2.5
2.75	21.31	1.668	1.292	27.5	24.06	1.747	1.321	31.8	2.75
3.	24.	1.780	1.334	32.	27.	1.864	1.365	36.9	3.
3.25	26.81	1.889	1.374	36.8	30.06	1.979	1.407	42.3	3.25
3.5	29.75	1.997	1.413	42.	33.25	2.091	1.446	48.1	3.5
3.75	32.81	2.103	1.450	47.6	36.56	2.201	1.483	54.2	3.75
4.	36.	2.207	1.486	53.5	40.	2.311	1.520	60.8	4.
4.5	42.75	2.412	1.533	65.5	47.25	2.523	1.589	75.1	4.5
5.	50.	2.612	1.616	80.8	55.	2.731	1.653	90.9	5.
6.	66.	3.004	1.733	114.4	72.	3.134	1.770	127.4	6.

Depth in Feet.	Bed 7 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 8 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	3.75	0.446	0.667	2.50	4.25	0.451	0.672	2.85	0.5
0.75	5.81	0.637	0.798	4.64	6.56	0.648	0.805	5.28	0.75
1.	8.	0.814	0.902	7.22	9.	0.831	0.911	8.2	1.
1.25	10.31	0.979	0.989	10.2	11.56	1.002	1.000	11.6	1.25
1.5	12.75	1.134	1.065	13.6	14.25	1.164	1.079	15.4	1.5
1.75	15.31	1.281	1.132	17.3	17.06	1.318	1.152	19.7	1.75
2.	18.	1.422	1.192	21.5	20.	1.464	1.210	24.2	2.
2.25	20.81	1.560	1.249	26.	23.06	1.606	1.267	29.2	2.25
2.5	23.75	1.688	1.300	30.9	26.25	1.742	1.320	34.7	2.5
2.75	26.81	1.815	1.347	36.1	29.56	1.873	1.368	40.4	2.75
3.	30.	1.938	1.392	41.8	33.	2.002	1.415	46.7	3.
3.25	33.31	2.057	1.434	47.8	35.56	2.069	1.439	51.2	3.25
3.5	36.75	2.169	1.473	54.1	40.25	2.269	1.506	60.6	3 5
3.75	40.31	2.290	1.513	61.	44.06	2.368	1.539	67.8	3.75
4.	44.	2.403	1.550	68.2	48.	2.486	1.577	75.7	4.
4.5	51.75	2.623	1.619	83.8	56.25	2.714	1.647	92.6	4.5
5.	60.	2.838	1.684	101.	65.	2.936	1 713	111.3	5.
6.	78.	3.254	1.804	140.7	84.	3.364	1.834	154.1	6.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 9 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 10 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	4.625	0.444	0.667	3.08	5.25	0.460	0.678	3.56	0.5
0.75	7.031	0.632	0.795	5.59	8.06	0.665	0.815	7.01	0.75
1.	10.	0.845	0.919	9.19	11.	0.858	0.926	10.2	1.
1.25	12.81	1.022	1.011	12.95	14.06	1.039	1.019	14.3	1.25
1.5	15.75	1.189	1.090	17.2	17.25	1.211	1.100	19.	1.5
1.75	18.81	1.349	1.161	21.8	20.56	1.375	1.173	24.1	1.
2.	22.	1.501	1.225	27.	24.	1.533	1.238	29.7	2.75
2.25	25.31	1.650	1.284	32.5	27.56	1.684	1.290	35.6	2.25
2.5	28.75	1.789	1.330	38.2	31.25	1.831	1.353	42.3	2.5
2.75	32.31	1.927	1.388	44.8	35.06	1.972	1.404	49.2	2.75
3.	36.	2.059	1.435	51.7	39.	2.110	1.452	56.6	3.
3.25	39.81	2.189	1.479	58.9	43.06	2.244	1.498	64.5	3.25
3.5	43.75	2.315	1.521	66.5	47.25	2.375	1.541	72.8	3.5
3.75	47.81	2.439	1.562	74.7	51.56	2.502	1.582	81.6	3.75
4.	52.	2.560	1.600	83.2	56.	2.628	1.621	90.8	4.
4.5	60.75	2.796	1.672	101.6	65.25	2.871	1.694	110.5	4.5
5.	70.	3.025	1.739	121.7	75.	3.107	1.763	132.2	5.
5.5	79.75	3.248	1.802	143.7	85.25	3.336	1.826	155.7	5.5
6.	90.	3.466	1.862	167.6	96.	3.560	1.887	181.2	6.

Depth in Feet.	Bed 11 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 12 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	5.625	0.453	0.674	3.79	6.25	0.466	0.682	4.26	0.5
0.75	8.531	0.643	0.802	6.84	9.56	0.677	0.823	7.87	0.75
1.	12.	0.868	0.932	11.2	13.	0.877	0.936	12.2	1.
1.25	15.31	1.053	1.026	15.7	16.56	1.066	1.032	17.1	1.25
1.5	18.75	1.230	1.109	20.8	20.25	1.246	1.116	22.6	1.5
1.75	22.31	1.399	1.183	26.4	24.06	1.420	1.192	28.7	1.75
2.	26.	1.561	1.249	32.5	28.	1.586	1.259	35.3	2.
2.25	29.81	1.719	1.311	39.1	32.06	1.746	1.321	42.4	2.25
2.5	33.75	1.868	1.367	46.1	36.25	1.901	1.379	50.	2.5
2.75	37.81	2.015	1.419	53.7	40.56	2.051	1.432	58.1	2.75
3.	42.	2.156	1.466	61.6	45.	2.197	1.482	66.7	3.
3.25	46.31	2.291	1.513	70.1	49.56	2.339	1.529	75.8	3.25
3.5	50.75	2.428	1.558	79.1	54.25	2.477	1.574	85.4	3.5
3.75	55.31	2.561	1.600	88.5	59.06	2.612	1.616	95.4	3.75
4.	60.	2.689	1.640	98.4	64.	2.745	1.657	106.	4.
4.5	69.75	2.940	1.715	119.6	74.25	3.003	1.733	128.7	4.5
5.	80.	3.182	1.784	142.7	85.	3.252	1.803	153.3	5.
5.5	90.75	3.417	1.848	167.7	96.25	3.493	1.869	179.9	5.5
6.	102.	3.647	1.910	194.8	108.	3.728	1.931	208.6	6.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet	Bed 13 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 14 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet
0.5	6.62	0.460	0.677	4.49	7.37	0.467	0.683	5.03	0.5
0.75	10.03	0.663	0.814	8.17	11.34	0.679	0.824	9.34	0.75
1.	14.	0.884	0.940	13.10	15.	0.891	0.944	14.2	1.
1.25	17.81	1.077	1.038	18.5	19.06	1.087	1.043	19.9	1.25
1.5	21.75	1.262	1.123	24.4	23.25	1.275	1.129	26.2	1.5
1.75	25.81	1.439	1.200	31.	27.56	1.454	1.206	33.2	1.75
2.	30.	1.608	1.268	38.	32.	1.628	1.276	40.8	2.
2.25	34.31	1.774	1.333	45.7	36.56	1.795	1.340	49.	2.25
2.5	38.75	1.931	1.382	53.6	41.25	1.958	1.398	57.7	2.5
2.75	43.31	2.085	1.444	62.5	46.06	2.115	1.454	67.	2.75
3.	48.	2.234	1.495	71.7	51.	2.268	1.506	76.8	3.
3.25	52.81	2.380	1.543	81.5	56.06	2.417	1.555	87.2	3.25
3.5	57.75	2.522	1.584	89.7	61.25	2.563	1.601	98.1	3.5
3.75	62.81	2.661	1.631	102.4	66.56	2.709	1.646	109.6	3.75
4.	68.	2.797	1.672	113.7	72.	2.845	1.687	121.5	4.
4.5	78.50	3.051	1.746	137.1	83.25	3.115	1.765	146.9	4.5
5.	90.	3.316	1.821	163.9	95.	3.376	1.810	171.9	5.
5.5	101.75	3.568	1.887	192.	107.25	3.630	1.905	204.3	5.5
6.	114.	3.814	1.950	222.3	120.	3.875	1.968	236.2	6.

Depth in Feet	Bed 1[illegible] Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 1[illegible] Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet
0.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.74	0.5
0.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	10.63	0.75
1.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	16.1	1.
1.25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	22.6	1.25
1.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	29.9	1.5
1.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	37.8	1.75
2.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	46.4	2.
2.25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	55.6	2.25
2.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	65.5	2.5
2.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	75.9	2.75
3.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	87.	3.
3.25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	98.6	3.25
3.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	110.8	3.5
3.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	123.5	3.75
4.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	136.9	4
4.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	165.3	4.5
5.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	195.9	5
5.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	228.9	5.5
6.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	264.[illegible]	6

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 17 Feet.				Bed 18 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	8.62	0.468	0.684	5.90	9.25	0.477	0.690	6.38	0.5
0.75	13.03	0.682	0.825	10.75	14.06	0.694	0.833	11.7	0.75
1.	18.	0.908	0.953	17.2	19.	0.912	0.955	18.1	1.
1.25	22.81	1.111	1.052	24.	24.06	1.117	1.057	25.4	1.25
1.5	27.75	1.306	1.143	31.7	29.25	1.315	1.147	33.5	1.5
1.75	32.81	1.495	1.222	40.1	34.56	1.506	1.227	42.4	1.75
2.	38.	1.677	1.295	49.2	40.	1.691	1.300	52.	2.
2.25	43.31	1.853	1.361	58.9	45.56	1.870	1.367	62.3	2.25
2.5	48.75	2.025	1.423	69.4	51.25	2.044	1.430	73.3	2.5
2.75	54.31	2.193	1.481	80.4	57.06	2.213	1.487	84.8	2.75
3.	60.	2.354	1.534	92.	63.	2.379	1.542	97.1	3.
3.25	65.81	2.513	1.585	104.3	69.06	2.541	1.594	110.1	3.25
3.5	71.75	2.667	1.633	117.2	75.25	2.697	1.642	123.6	3.5
3.75	77.81	2.819	1.679	130.6	81.56	2.851	1.688	137.7	3.75
4.	84.	2.967	1.722	144.6	88.	3.002	1.733	152.5	4.
4.5	96.75	3.255	1.804	174.5	101.25	3.296	1.810	183.8	4.5
5.	110.	3.532	1.880	206.8	115.	3.578	1.891	217.5	5.
5.5	123.75	3.801	1.950	241.3	129.25	3.852	1.962	253.6	5.5
6.	138.	4.062	2.015	278.1	144.	4.117	2.029	292.2	6.
7.	168.	4.565	2.137	359.	175.	4.630	2.152	376.6	7.

	Bed 19 Feet.				Bed 20 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	9.62	0.471	0.686	6.60	10.25	0.479	0.692	7.09	0.5
0.75	14.53	0.688	0.830	12.1	15.56	0.704	0.839	13.1	0.75
1.	20.	0.876	0.936	18.7	21.	0.920	0.959	20.1	1.
1.25	25.31	1.123	1.060	26.8	26.56	1.129	1.063	28.2	1.25
1.5	30.75	1.323	1.150	35.4	32.25	1.330	1.153	37.2	1.5
1.75	36.31	1.516	1.231	44.7	38.06	1.525	1.235	47.	1.75
2.	42.	1.703	1.305	54.8	44.	1.715	1.309	57.6	2.
2.25	47.81	1.886	1.373	65.6	50.06	1.898	1.377	68.9	2.25
2.5	53.75	2.062	1.436	77.2	56.25	2.078	1.442	81.1	2.5
2.75	59.81	2.234	1.494	89.4	62.56	2.252	1.501	93.9	2.75
3.	66.	2.401	1.550	102.3	69.	2.422	1.556	107.4	3.
3.25	72.31	2.565	1.601	115.8	75.56	2.589	1.609	121.6	3.25
3.5	78.75	2.725	1.651	130.	82.25	2.751	1.659	136.5	3.5
3.75	88.31	2.882	1.700	150.1	89.06	2.998	1.731	154.2	3.75
4.	92.	3.035	1.742	160.3	96.	3.066	1.751	168.1	4.
4.5	105.75	3.333	1.825	193.	110.25	3.369	1.835	202.3	4.5
5.	120.	3.621	1.903	228.4	125.	3.661	1.913	239.1	5.
5.5	134.75	3.899	1.975	266.1	140.25	3.944	1.986	278.5	5.5
6.	150.	4.170	2.042	306.3	156.	4.220	2.054	320.4	6.
7.	182.	4.691	2.166	394.2	189.	4.748	2.179	411.8	7.
8.	216.	5.189	2.276	491.6	224.	5.255	2.292	513.4	8.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 25 Feet.				Bed 30 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	12.25	0.464	.681	8.34	15.25	0.486	.697	10.63	0.5
0.75	19.31	0.712	.844	16.3	23.06	0.718	.847	19.5	0.75
1.	26.	0.934	.966	25.1	31.	0.944	.976	30.3	1.
1.25	32.81	1.150	1.072	35.2	39.06	1.165	1.079	42.1	1.25
1.5	39.75	1.359	1.166	46.3	47.25	1.380	1.175	55.5	1.5
1.75	46.81	1.563	1.250	58.5	55.56	1.592	1.261	70.1	1.75
2.	54.	1.761	1.327	71.7	64.	1.795	1.340	85.8	2.
2.25	61.31	1.954	1.397	85.6	72.56	1.995	1.412	102.5	2.25
2.5	68.75	2.144	1.464	100.7	81.25	2.172	1.474	119.8	2.5
2.75	76.31	2.328	1.526	116.4	90.06	2.384	1.544	139.1	2.75
3.	84.	2.509	1.584	133.1	99.	2.573	1.604	158.8	3.
3.25	91.81	2.684	1.639	150.5	108.06	2.758	1.661	179.5	3.25
3.5	99.75	2.858	1.691	168.7	117.25	2.939	1.711	200.6	3.5
3.75	107.81	3.028	1.740	187.6	126.56	3.141	1.772	224.3	3.75
4.	116.	3.193	1.787	207.3	136.	3.291	1.814	246.7	4.
4.25	124.31	3.358	1.832	227.7	145.56	3.464	1.861	270.9	4.25
4.5	132.75	3.519	1.876	249.	155.25	3.633	1.906	295.9	4.5
4.75	141.31	3.677	1.917	270.9	165.06	3.800	1.949	321.7	4.75
5.	150.	3.831	1.957	293.6	175.	3.965	1.991	348.4	5.
5.25	158.81	3.985	1.971	313.	185.06	4.126	2.031	375.9	5.25
5.5	167.75	4.136	2.034	341.2	195.25	4.286	2.070	404.2	5.5
5.75	176.81	4.285	2.070	366.	205.56	4.443	2.108	433.3	5.75
6.	186.	4.432	2.105	391.5	216.	4.599	2.145	463.3	6.
6.25	195.31	4.576	2.139	417.8	226.56	4.752	2.179	493.7	6.25
6.5	204.75	4.720	2.172	444.7	237.25	4.903	2.214	525.3	6.5
6.75	214.31	4.861	2.205	472.6	248.06	5.053	2.248	557.6	6.75
7.	224.	5.	2.236	500.9	259.	5.201	2.281	590.8	7.
7.25	233.81	5.138	2.267	530.	270.06	5.347	2.312	624.4	7.25
7.5	243.75	5.274	2.296	559.7	281.25	5.492	2.344	659.2	7.5
7.75	253.81	5.409	2.325	590.1	292.56	5.635	2.374	694.5	7.75
8.	264.	5.541	2.354	621.5	304.	5.776	2.403	730.5	8.
8.25	274.31	5.675	2.382	653.4	315.56	5.917	2.432	767.4	8.25
8.5	284.75	5.806	2.408	685.7	327.25	6.055	2.460	805.	8.5
8.75	295.31	5.936	2.436	719.4	339.06	6.193	2.488	843.6	8.75
9.	306.	6.065	2.463	753.7	351.	6.329	2.515	882.8	9.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 35 Feet. a	Bed 35 Feet. r	Bed 35 Feet. $\sqrt{r}$	Bed 35 Feet. $a\sqrt{r}$	Bed 40 Feet. a	Bed 40 Feet. r	Bed 40 Feet. $\sqrt{r}$	Bed 40 Feet. $a\sqrt{r}$	Depth in Feet.
0.75	23.06	0.621	.788	18.2	30.56	0.726	.852	26.	0.75
1.	36.	0.952	.976	35.1	41.	0.957	.978	40.1	1.
1.25	45.31	1.176	1.082	49.	51.56	1.184	1.088	56.1	1.25
1.5	54.75	1.395	1.181	64.7	62.25	1.407	1.190	74.1	1.5
1.75	64.30	1.610	1.269	81.6	73.06	1.625	1.275	93.2	1.75
2.	74.	1.820	1.349	99.8	84.	1.840	1.356	113.9	2.
2.25	83.81	2.026	1.423	119.3	95.06	2.050	1.432	136.1	2.25
2.5	93.75	2.228	1.493	140.	106.25	2.257	1.502	159.6	2.5
2.75	103.81	2.426	1.557	161.6	117.56	2.460	1.568	184.3	2.75
3.	114.	2.622	1.619	184.6	129.	2.661	1.631	210.4	3.
3.25	124.31	2.815	1.678	208.6	140.56	2.838	1.685	236.8	3.25
3.5	134.75	3.001	1.732	233.4	152.25	3.051	1.747	266.	3.5
3.75	145.31	3.197	1.788	259.8	164.06	3.242	1.801	295.5	3.75
4.	156.	3.368	1.835	286.3	176.	3.431	1.852	326.	4.
4.25	166.81	3.547	1.883	314.1	188.06	3.615	1.901	357.5	4.25
4.5	177.75	3.724	1.930	343.1	200.25	3.798	1.949	390.3	4.5
4.75	188.81	3.898	1.974	372.7	212.56	3.977	1.994	423.8	4.75
5.	200.	4.070	2.017	403.4	225.	4.155	2.038	458.6	5.
5.25	211.31	4.239	2.059	435.1	237.56	4.331	2.081	494.4	5.25
5.5	222.75	4.406	2.099	467.6	250.25	4.504	2.122	531.	5.5
5.75	234.31	4.571	2.138	501.	263.06	4.676	2.162	567.7	5.75
6.	246.	4.733	2.176	535.3	276.	4.844	2.201	607.5	6.
6.25	257.81	4.894	2.212	570.3	289.26	5.015	2.239	647.7	6.25
6.5	269.75	5.053	2.248	606.4	302.25	5.177	2.275	687.6	6.5
6.75	281.81	5.206	2.282	643.1	315.56	5.340	2.311	729.3	6.75
7.	294.	5.365	2.316	680.9	329.	5.501	2.343	770.8	7.
7.25	306.21	5.517	2.349	719.3	342.56	5.661	2.379	815.	7.25
7.5	318.75	5.671	2.381	758.9	356.25	5.830	2.414	860.	7.5
7.75	331.31	5.821	2.416	800.4	370.06	5.976	2.444	904.4	7.75
8.	344.	5.968	2.443	840.4	384.	6.132	2.476	950.8	8.
8.25	356.81	6.117	2.473	882.4	398.06	6.285	2.507	997.9	8.25
8.5	369.75	6.262	2.502	925.1	412.25	6.437	2.537	1046.	8.5
8.75	382.81	6.407	2.531	968.9	426.56	6.588	2.566	1095.	8.75
9.	396.	6.550	2.559	1013.	441.	6.737	2.596	1145.	9.
9.5	422.75	6.833	2.614	1105.	470.25	7.107	2.666	1254.	9.5
10.	450.	7.111	2.666	1200.	500.	7.322	2.706	1353.	10.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formula

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	BED 45 FEET.				BED 50 FEET.				Depth in Feet.
	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	
1	46	0.982	0.99[illegible]	[illegible]	51	.964	0.982	50.1	1.
1.5	[illegible]	1.416	1.19[illegible]	[illegible]	77.25	1.424	1.193	92.2	1.5
1.75	[illegible]	1.[illegible]	1.2[illegible]	[illegible]	90.56	1.648	1.284	116.3	1.75
2	94	1.[illegible]	1.382	[illegible]	104	1.[illegible]	1.367	142.2	2
2.25	[illegible]	2.[illegible]	1.4[illegible]	[illegible]	[illegible]	2.[illegible]	1.444	[illegible]	2.25
2.5	[illegible]	2.[illegible]	1.5[illegible]	[illegible]	[illegible]	2.300	1.5[illegible]	[illegible]	2.5
2.75	[illegible]	2.4[illegible]	1.5[illegible]	[illegible]	[illegible]	2.[illegible]	1.5[illegible]	[illegible]	2.75
3	[illegible]	2.[illegible]	1.[illegible]	[illegible]	[illegible]	2.[illegible]	1.[illegible]	262.2	3
3.25	[illegible]	2.[illegible]	1.[illegible]	[illegible]	[illegible]	2.[illegible]	1.[illegible]	[illegible]	3.25
3.5	[illegible]	3.[illegible]	1.[illegible]	[illegible]	[illegible]	3.[illegible]	1.[illegible]	[illegible]	3.5
3.75	[illegible]	3.[illegible]	1.[illegible]	[illegible]	[illegible]	3.[illegible]	1.[illegible]	[illegible]	3.75
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4
4.25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.25
4.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.5
4.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 60 Feet.				Bed 70 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	61.	0.971	0.985	60.1	71.	0.975	0.987	70.1	1.
1.5	92.25	1.436	1.199	110.6	107.25	1.445	1.200	128.7	1.5
2.	124.	1.889	1.377	170.7	144.	1.903	1.346	193.8	2.
2.25	140.06	2.110	1.452	203.4	162.56	2.129	1.459	237.2	2.25
2.5	156.25	2.330	1.526	238.4	181.25	2.352	1.534	278.	2.5
2.75	172.56	2.546	1.595	275.2	200.06	2.572	1.604	320.9	2.75
3.	189.	2.760	1.661	313.9	219.	2.790	1.670	365.7	3.
3.25	205.56	2.971	1.724	355.2	238.06	3.006	1.734	412.8	3.25
3.5	222.25	3.180	1.783	396.3	257.25	3.220	1.794	461.5	3.5
3.75	239.06	3.386	1.838	439.4	276.56	3.431	1.852	512.2	3.75
4.	256.	3.590	1.895	475.1	296.	3.640	1.908	564.8	4.
4.25	273.06	3.791	1.947	531.6	315.56	3.847	1.961	618.8	4.25
4.5	290.25	3.991	1.998	579.9	335.25	4.052	2.013	674.9	4.5
4.75	307.56	4.188	2.046	629.3	355.06	4.256	2.063	732.5	4.75
5.	325.	4.384	2.095	680.9	375.	4.457	2.111	791.6	5.
5.25	342.56	4.577	2.139	732.7	395.06	4.656	2.158	852.5	5.25
5.5	360.25	4.768	2.183	786.4	415.25	4.858	2.204	915.2	5.5
5.75	378.06	4.957	2.226	841.6	435.56	5.049	2.247	978.7	5.75
6.	396.	5.145	2.268	898.1	456.	5.243	2.289	1043.8	6.
6.25	414.06	5.330	2.309	956.1	476.56	5.435	2.331	1110.9	6.25
6.5	432.25	5.515	2.348	1014.9	497.25	5.626	2.372	1179.5	6.5
6.75	450.56	5.697	2.387	1075.5	518.06	5.815	2.411	1249.	6.75
7.	469.	5.877	2.424	1136.8	539.	6.002	2.450	1320.6	7.
7.25	487.56	6.056	2.461	1199.9	560.06	6.188	2.487	1392.9	7.25
7.5	506.25	6.234	2.497	1264.1	581.25	6.373	2.524	1467.1	7.5
7.75	525.06	6.409	2.531	1328.9	602.56	6.555	2.560	1542.6	7.75
8.	544.	6.584	2.566	1396.	624.	6.736	2.596	1619.9	8.
8.25	563.06	6.757	2.599	1463.4	645.56	6.917	2.630	1697.8	8.25
8.5	582.25	6.928	2.632	1532.5	667.25	7.095	2.664	1777.6	8.5
8.75	601.56	7.098	2.664	1602.6	689.06	7.272	2.696	1857.7	8.75
9.	621.	7.267	2.696	1674.2	711.	7.448	2.729	1940.3	9.
9.5	660.25	7.600	2.759	1821.6	755.25	7.797	2.790	2107.1	9.5
10.	700.	7.929	2.816	1971.2	800.	8.140	2.853	2282.4	10.
10.5	740.25	8.253	2.873	2126.7	845.25	8.478	2.912	2461.4	10.5
11.	781.	8.572	2.928	2286.8	891.	8.812	2.968	2644.5	11.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 80 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 90 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	81.	0.978	.989	80.1	91.	0.980	.990	90.1	1.
2.	164.	1.915	1.384	227.0	184.	1.923	1.387	255.2	2.
2.25	185.06	2.143	1.464	270.9	207.56	2.154	1.467	304.5	2.25
2.5	206.25	2.369	1.539	317.4	231.25	2.382	1.543	356.8	2.5
2.75	227.56	2.592	1.610	366.4	255.06	2.609	1.612	411.2	2.75
3.	249.	2.814	1.678	417.8	279.	2.833	1.683	469.6	3.
3.25	270.56	3.034	1.742	471.3	303.06	3.055	1.748	529.7	3.25
3.5	292.25	3.251	1.803	526.9	327.25	3.276	1.810	592.3	3.5
3.75	314.06	3.466	1.862	584.8	351.56	3.494	1.869	657.1	3.75
4.	336.	3.680	1.918	644.4	376.	3.711	1.926	724.2	4.
4.25	358.06	3.891	1.973	706.5	400.56	3.926	1.981	793.5	4.25
4.5	380.25	4.101	2.025	770.	425.25	4.139	2.034	865.	4.5
4.75	402.56	4.308	2.076	835.7	450.06	4.351	2.086	938.8	4.75
5.	425.	4.514	2.125	903.1	475.	4.562	2.136	1015.	5.
5.25	447.56	4.719	2.172	972.1	500.06	4.769	2.184	1092.	5.25
5.5	470.25	4.921	2.218	1043.	525.25	4.976	2.231	1172.	5.5
5.75	493.06	5.122	2.263	1116.	550.56	5.181	2.276	1253.	5.75
6.	516.	5.321	2.307	1190.	576.	5.397	2.320	1336.	6.
6.25	539.06	5.519	2.349	1266.	601.56	5.587	2.364	1455.	6.25
6.5	562.25	5.715	2.391	1344.	627.25	5.788	2.406	1509.	6.5
6.75	585.56	5.909	2.431	1423.	653.06	5.986	2.446	1597.	6.75
7.	609.	6.102	2.470	1504.	679.	6.184	2.487	1689.	7.
7.25	632.56	6.293	2.508	1586.	705.06	6.380	2.526	1781.	7.25
7.5	656.25	6.484	2.546	1671.	731.25	6.575	2.564	1875.	7.5
7.75	680.06	6.672	2.583	1757.	757.56	6.769	2.602	1971.	7.75
8.	704.	6.860	2.619	1844.	784.	6.961	2.638	2068.	8.
8.25	728.06	7.046	2.654	1932.	810.56	7.152	2.674	2167.	8.25
8.5	752.25	7.230	2.689	2023.	837.25	7.342	2.710	2269.	8.5
8.75	776.56	7.414	2.723	2115.	864.06	7.530	2.744	2371.	8.75
9.	801.	7.595	2.756	2208.	891.	7.717	2.778	2475.	9.
9.25	825.56	7.777	2.789	2302.	918.06	7.903	2.811	2581.	9.25
9.5	850.25	7.956	2.821	2399.	945.25	8.088	2.844	2688.	9.5
9.75	875.06	8.134	2.852	2496.	972.56	8.271	2.876	2797.	9.75
10.	900.	8.312	2.883	2595.	1000.	8.454	2.907	2907.	10.
10.5	950.25	8.663	2.943	2797.	1055.25	8.816	2.969	3133.	10.5
11.	1001.	9.009	3.001	3004.	1111.	9.173	3.028	3364.	11.
12.	1104.	9.689	3.113	3437.	1224.	9.876	3.142	3846.	12.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 100 Feet.				Bed 120 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	101.	0.982	0.991	100.1	121.	0.985	0.992	120.	1.
2.	204.	1.931	1.389	283.4	244.	1.942	1.393	339.9	2.
2.25	230.06	2.163	1.470	338.2	275.06	2.177	1.475	405.7	2.25
2.5	256.25	2.393	1.546	396.2	306.25	2.410	1.552	475.3	2.5
2.75	282.56	2.622	1.619	457.5	337.56	2.642	1.625	548.5	2.75
3.	309.	2.848	1.687	521.3	369.	2.872	1.695	625.5	3.
3.25	335.56	3.073	1.752	587.9	400.56	3.101	1.761	705.4	3.25
3.5	362.25	3.296	1.816	657.8	432.25	3.328	1.824	788.4	3.5
3.75	389.06	3.517	1.875	729.5	464.06	3.553	1.885	874.8	3.75
4.	416.	3.737	1.933	804.1	496.	3.777	1.943	963.7	4.
4.25	443.06	3.955	1.988	880.8	528.06	4.	2.	1056.	4.25
4.5	470.25	4.171	2.042	960.3	560.25	4.221	2.054	1151.	4.5
4.75	497.56	4.386	2.094	1042.	592.56	4.441	2.107	1249.	4.75
5.	525.	4.600	2.145	1126.	625.	4.659	2.158	1349.	5.
5.25	552.56	4.811	2.193	1212.	657.56	4.876	2.208	1452.	5.25
5.5	580.25	5.021	2.241	1300.	690.25	5.092	2.256	1557.	5.5
5.75	608.06	5.230	2.287	1391.	723.06	5.306	2.303	1665.	5.75
6.	636.	5.437	2.331	1483.	756.	5.519	2.349	1776.	6.
6.25	664.06	5.643	2.375	1577.	789.06	5.731	2.394	1889.	6.25
6.5	692.25	5.848	2.418	1674.	822.25	5.942	2.437	2004.	6.5
6.75	720.56	6.050	2.460	1773.	855.56	6.151	2.480	2122.	6.75
7.	749.	6.252	2.500	1873.	889.	6.359	2.521	2241.	7.
7.25	777.56	6.452	2.540	1957.	922.56	6.566	2.562	2364.	7.25
7.5	806.25	6.652	2.579	2079.	956.25	6.772	2.602	2488.	7.5
7.75	835.06	6.849	2.617	2185.	990.06	6.976	2.641	2615.	7.75
8.	864.	7.046	2.654	2293.	1024.	7.179	2.679	2743.	8.
8.25	893.06	7.241	2.691	2403.	1058.06	7.382	2.717	2875.	8.25
8.5	922.25	7.435	2.726	2514.	1092.25	7.583	2.753	3007.	8.5
8.75	951.56	7.628	2.762	2628.	1126.56	7.783	2.790	3143.	8.75
9.	981.	7.819	2.796	2743.	1161.	7.982	2.825	3280.	9.
9.25	1010.56	8.010	2.830	2860.	1195.56	8.180	2.860	3419.	9.25
9.5	1040.25	8.199	2.863	2978.	1230.25	8.376	2.894	3560.	9.5
9.75	1070.06	8.387	2.896	3099.	1265.06	8.572	2.928	3704.	9.75
10.	1100.	8.575	2.928	3221.	1300.	8.767	2.961	3849.	10.
10.5	1160.25	8.946	2.991	3470.	1370.25	9.153	3.025	4145.	10.5
11.	1221.	9.313	3.051	3725.	1441.	9.536	3.088	4450.	11.
11.5	1282.25	9.675	3.110	3988.	1512.25	9.915	3.149	4762.	11.5
12.	1344.	10.03	3.167	4256.	1584.	10.29	3.208	5081.	12.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 140 Feet.				Bed 160 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	141.	0.987	0.993	140.	161.	0.989	0.994	160.	1.
2.	284.	1.950	1.396	396.5	324.	1.956	1.398	453.	2.
2.25	320.06	2.187	1.465	468.9	365.06	2.194	1.481	540.7	2.25
2.5	356.25	2.422	1.556	554.3	406.25	2.432	1.559	633.3	2.5
2.75	392.56	2.656	1.630	639.9	447.56	2.668	1.639	733.6	2.75
3.	429.	2.889	1.699	728.9	489.	2.902	1.704	833.3	3.
3.25	465.56	3.121	1.767	822.6	530.56	3.136	1.771	939.6	3.25
3.5	502.25	3.351	1.831	906.1	572.25	3.368	1.835	1050.	3.5
3.75	539.06	3.579	1.892	1020.	614.06	3.599	1.897	1165.	3.75
4.	576.	3.807	1.951	1124.	656.	3.829	1.957	1284.	4.
4.25	613.06	4.033	2.008	1231.	698.06	4.058	2.014	1406.	4.25
4.5	650.25	4.258	2.063	1341.	740.25	4.286	2.070	1532.	4.5
4.75	687.56	4.481	2.117	1456.	782.56	4.512	2.124	1662.	4.75
5.	725.	4.703	2.169	1573.	825.	4.738	2.177	1796.	5.
5.25	762.56	4.924	2.219	1692.	867.56	4.962	2.228	1933.	5.25
5.5	800.25	5.144	2.268	1815.	910.25	5.185	2.277	2073.	5.5
5.75	838.06	5.363	2.315	1940.	953.06	5.407	2.325	2216.	5.75
6.	876.	5.581	2.362	2069.	996.	5.628	2.372	2363.	6.
6.25	914.06	5.797	2.408	2201.	1039.06	5.848	2.418	2512.	6.25
6.5	952.25	6.013	2.452	2335.	1082.25	6.067	2.463	2666.	6.5
6.75	990.56	6.226	2.495	2471.	1125.56	6.285	2.507	2822.	6.75
7.	1029.	6.439	2.538	2612.	1169.	6.498	2.549	2980.	7.
7.25	1067.56	6.651	2.579	2753.	1212.56	6.717	2.592	3143.	7.25
7.5	1106.25	6.862	2.620	2898.	1256.25	6.927	2.632	3306.	7.5
7.75	1145.06	7.072	2.659	3045.	1300.06	7.146	2.673	3475.	7.75
8.	1184.	7.280	2.700	3197.	1344.	7.359	2.713	3646.	8.
8.25	1223.06	7.488	2.736	3346.	1386.06	7.561	2.750	3812.	8.25
8.5	1262.25	7.695	2.774	3501.	1432.25	7.782	2.790	3996.	8.5
8.75	1301.56	7.900	2.811	3659.	1476.56	7.992	2.827	4174.	8.75
9.	1341.	8.105	2.847	3818.	1521.	8.201	2.864	4356.	9.
9.25	1380.56	8.289	2.882	3979.	1565.56	8.410	2.900	4540.	9.25
9.5	1420.25	8.511	2.917	4143.	1610.25	8.617	2.936	4728.	9.5
9.75	1460.06	8.713	2.952	4310.	1655.06	8.823	2.970	4916.	9.75
10.	1500.	8.912	2.985	4478.	1700.	9.029	3.005	5109.	10.
10.5	1580.25	9.312	3.051	4821.	1790.25	9.437	3.072	5499.	10.5
11.	1661.	9.707	3.116	5176.	1881.	9.843	3.137	5901.	11.
11.5	1742.25	10.098	3.178	5537.	1972.25	10.24	3.200	6311.	11.5
12.	1824.	10.49	3.249	5926.	2064.	10.64	3.262	6733.	12.
13.	1989.	11.252	3.354	6671.	2249.	11.43	3.381	7604.	13.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 180 Feet.				Bed 200 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	181.	0.990	0.995	180.1	201.	0.991	0.995	200.	1.
2.	364.	1.961	1.400	509.6	404.	1.964	1.402	566.4	2.
2.5	456.25	2.439	1.562	712.7	506.25	2.445	1.564	791.8	2.5
2.75	502.56	2.676	1.636	822.2	557.56	2.683	1.638	913.3	2.75
3.	549.	2.913	1.706	936.6	609.	2.921	1.709	1041.	3.
3.25	595.56	3.148	1.774	1057.	660.56	3.158	1.777	1174.	3.25
3.5	642.25	3.382	1.839	1181.	712.25	3.393	1.842	1312.	3.5
3.75	689.06	3.615	1.901	1310.	764.06	3.628	1.905	1456.	3.75
4.	736.	3.847	1.961	1443.	816.	3.862	1.965	1603.	4.
4.25	783.06	4.078	2.019	1581.	868.06	4.094	2.023	1756.	4.25
4.5	830.25	4.308	2.075	1723.	920.25	4.326	2.080	1914.	4.5
4.75	877.56	4.537	2.130	1869.	972.56	4.557	2.134	2075.	4.75
5.	925.	4.765	2.183	2019.	1025.	4.787	2.188	2243.	5.
5.25	972.56	4.991	2.234	2173.	1077.56	5.015	2.239	2413.	5.25
5.5	1020.25	5.217	2.284	2330.	1130.25	5.243	2.290	2588.	5.5
5.75	1068.06	5.442	2.333	2492.	1183.06	5.470	2.339	2767.	5.75
6.	1116.	5.666	2.380	2656.	1236.	5.697	2.387	2950.	6.
6.25	1164.06	5.889	2.427	2825.	1289.06	5.921	2.433	3136.	6.25
6.5	1212.25	6.111	2.472	2997.	1342.25	6.146	2.479	3327.	6.5
6.75	1260.56	6.332	2.516	3172.	1395.56	6.370	2.524	3522.	6.75
7.	1309.	6.552	2.560	3351.	1449.	6.592	2.567	3720.	7.
7.25	1357.56	6.770	2.602	3532.	1502.56	6.814	2.610	3922.	7.25
7.5	1406.25	6.973	2.641	3714.	1556.25	7.035	2.652	4127.	7.5
7.75	1455.06	7.206	2.684	3905.	1610.06	7.255	2.693	4336.	7.75
8.	1504.	7.422	2.724	4097.	1664.	7.474	2.734	4549.	8.
8.25	1553.06	7.638	2.763	4291.	1718.06	7.693	2.773	4764.	8.25
8.5	1602.25	7.853	2.802	4490.	1772.25	7.910	2.812	4984.	8.5
8.75	1651.56	8.066	2.840	4690.	1826.56	8.127	2.851	5208.	8.75
9.	1701.	8.279	2.877	4920.	1881.	8.343	2.888	5432.	9.
9.25	1750.56	8.491	2.914	5101.	1935.56	8.558	2.925	5662.	9.25
9.5	1800.25	8.702	2.950	5311.	1990.25	8.773	2.962	5895.	9.5
9.75	1850.	8.913	2.985	5522.	2045.	8.986	2.997	6129.	9.75
10.	1900.	9.122	3.020	5738.	2100.	9.199	3.033	6369.	10.
10.5	2000.	9.539	3.089	6178.	2210.	9.622	3.102	6855.	10.5
11.	2101.	9.952	3.154	6627.	2321.	10.04	3.169	7355.	11.
11.5	2202.	10.36	3.220	7091.	2432.	10.46	3.234	7865.	11.5
12.	2304.	10.77	3.282	7562.	2544.	10.87	3.298	8390.	12.
13.	2509.	11.59	3.406	8546.	2769.	11.69	3.417	9462.	13.
14.	2716.	12.37	3.517	9552.	2996.	12.50	3.536	10594.	14.

TABLE [illegible].

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $c\sqrt{r}$ for use in the formula

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } q = a\, c\sqrt{r} \times \sqrt{s}$$

	Bed [illegible] Feet				Bed [illegible] Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$c\sqrt{r}$	a	r	$\sqrt{r}$	$c\sqrt{r}$	Depth in Feet.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 260 Feet.				Bed 280 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
2.	524.	1.972	1.404	735.7	564.	1.974	1.405	792.4	2.
2.5	656.25	2.457	1.567	1028.	706.25	2.460	1.568	1107.	2.5
3.	789.	2.939	1.714	1352.	849.	2.943	1.716	1457.	3.
3.25	855.56	3.178	1.783	1525.	920.56	3.183	1.784	1642.	3.25
3.5	922.25	3.417	1.849	1705.	992.25	3.423	1.850	1836.	3.5
3.75	989.06	3.655	1.912	1891.	1064.06	3.662	1.914	2037.	3.75
4.	1056.	3.892	1.973	2083.	1136.	3.900	1.977	2246.	4.
4.25	1123.06	4.129	2.032	2282.	1208.06	4.136	2.034	2457.	4.25
4.5	1190.25	4.364	2.089	2486.	1280.25	4.373	2.091	2677.	4.5
4.75	1257.56	4.599	2.145	2697.	1352.56	4.610	2.147	2904.	4.75
5.	1325.	4.833	2.198	2912.	1425.	4.845	2.201	3136.	5.
5.25	1392.56	5.067	2.251	3135.	1497.56	5.079	2.254	3376.	5.25
5.5	1460.25	5.299	2.302	3361.	1570.25	5.313	2.305	3619.	5.5
5.75	1528.06	5.531	2.352	3594.	1643.06	5.546	2.355	3869.	5.75
6.	1596.	5.762	2.400	3830.	1716.	5.778	2.404	4125.	6.
6.25	1664.06	5.993	2.448	4074.	1789.06	6.010	2.452	4387.	6.25
6.5	1732.25	6.223	2.494	4320.	1862.25	6.241	2.498	4652.	6.5
6.75	1800.56	6.452	2.541	4575.	1835.56	6.470	2.544	4670.	6.75
7.	1869.	6.680	2.585	4831.	2009.	6.701	2.589	5201.	7.
7.25	1937.56	6.908	2.628	5092.	2082.56	6.930	2.632	5481.	7.25
7.5	2006.25	7.134	2.671	5359.	2156.25	7.159	2.676	5770.	7.5
7.75	2075.06	7.361	2.713	5630.	2230.06	7.386	2.718	6061.	7.75
8.	2144.	7.586	2.754	5905.	2304.	7.613	2.759	6357.	8.
8.25	2213.06	7.811	2.795	6186.	2378.06	7.840	2.800	6659.	8.25
8.5	2282.25	8.035	2.835	6470.	2452.25	8.066	2.840	6964.	8.5
8.75	2351.56	8.258	2.874	6758.	2526.56	8.290	2.879	7274.	8.75
9.	2421.	8.481	2.912	7050.	2601.	8.515	2.918	7590.	9.
9.25	2490.56	8.703	2.950	7347.	2675.56	8.739	2.956	7909.	9.25
9.5	2560.25	8.925	2.987	7647.	2750.25	8.962	2.993	8231.	9.5
9.75	2630.06	9.146	3.024	7953.	2825.06	9.185	3.031	8563.	9.75
10.	2700.	9.366	3.060	8262.	2900.	9.407	3.067	8894.	10.
10.5	2840.25	9.804	3.131	8893.	3050.25	9.849	3.138	9572.	10.5
11.	2981.	10.24	3.200	9539.	3201.	10.29	3.203	10253.	11.
11.5	3122.25	10.67	3.266	10197.	3352.25	10.73	3.276	10982.	11.5
12.	3264.	11.10	3.332	10876.	3504.	11.16	3.341	11707.	12.
13.	3549.	11.96	3.458	12272.	3809.	12.02	3.467	13206.	13.
14.	3836.	12.80	3.578	13725.	4116.	12.88	3.589	14772.	14.
15.	4125.	13.64	3.693	15234.	4425.	13.72	3.705	16395.	15.
16.	4416.	14.47	3.804	16798.	4736.	14.56	3.815	18068.	16.
18.	5004.	16.09	4.012	20076.	5364.	16.21	4.026	21595.	18.

TABLE 8.

Channels having a trapezoidal section, with side slopes of 1 to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$[illegible] \text{ and } Q = [illegible] \times a\sqrt{r} \times \sqrt{s}$$

BED 300 FEET.

Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$
2	604	[illegible]	[illegible]	[illegible]
2.5	756.25	2.463	[illegible]	[illegible]
3.	909	[illegible]	[illegible]	[illegible]
3.25	985.56	[illegible]	[illegible]	[illegible]
3.5	1062.25	[illegible]	[illegible]	[illegible]
3.75	1139.06	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]
4.25	[illegible]	[illegible]	[illegible]	[illegible]
4.5	[illegible]	[illegible]	[illegible]	[illegible]
4.75	[illegible]	[illegible]	[illegible]	[illegible]
5.	[illegible]	[illegible]	[illegible]	[illegible]
5.25	[illegible]	[illegible]	[illegible]	[illegible]
5.5	[illegible]	[illegible]	[illegible]	[illegible]
5.75	[illegible]	[illegible]	[illegible]	[illegible]
6.	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 1 Foot.				Bed 2 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	0.62	0.293	0.54	0.34	1.12	0.359	0.60	0.68	0.5
0.75	1.03	0.385	0.62	0.64	1.78	0.484	0.69	1.24	0.75
1.	1.50	0.464	0.68	1.02	2.50	0.590	0.77	1.92	1.
1.25	2.03	0.535	0.73	1.48	3.28	0.684	0.83	2.71	1.25
1.5	2.62	0.602	0.78	2.04	4.12	0.770	0.88	3.62	1.5
1.75	3.28	0.668	0.82	2.69	5.03	0.851	0.92	4.64	1.75
2.	4.00	0.731	0.86	3.43	6.00	0.927	0.96	5.78	2.
2.25					7.03	1.000	1.00	7.03	2.25
2.5					8.12	1.070	1.03	8.41	2.5
2.75					9.28	1.139	1.07	9.90	2.75
3.					10.50	1.217	1.10	11.5	3.
3.25					11.78	1.271	1.13	13.3	3.25
3.5					13.12	1.337	1.16	15.2	3.5
3.75					14.53	1.399	1.18	17.2	3.75
4.					16.00	1.462	1.21	19.4	4.

	Bed 3 Feet.				Bed 4 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	1.62	0.394	0.63	1.02	2.12	0.411	0.64	1.37	0.5
0.75	2.53	0.541	0.73	1.87	3.28	0.578	0.76	2.50	0.75
1.	3.50	0.668	0.82	2.86	4.50	0.722	0.85	3.82	1.
1.25	4.53	0.782	0.88	4.00	5.78	0.851	0.92	5.33	1.25
1.5	5.62	0.885	0.94	5.29	7.35	0.969	0.98	7.01	1.5
1.75	6.78	0.981	0.99	6.72	8.53	1.078	1.04	8.86	1.75
2.	8.00	1.071	1.03	8.28	10.00	1.180	1.09	10.9	2.
2.25	9.28	1.156	1.07	9.98	11.53	1.277	1.13	13.0	2.25
2.5	10.62	1.237	1.11	11.8	13.12	1.369	1.17	15.4	2.5
2.75	12.03	1.315	1.15	13.8	14.78	1.456	1.21	17.8	2.75
3.	13.50	1.391	1.18	15.9	16.50	1.541	1.24	20.5	3.
3.25	15.03	1.464	1.21	18.2	18.28	1.623	1.27	23.3	3.25
3.5	16.62	1.536	1.24	20.6	20.12	1.702	1.30	26.3	3.5
3.75	18.28	1.606	1.27	23.2	22.03	1.779	1.33	29.4	3.75
4.	20.00	1.675	1.29	25.9	24.00	1.854	1.36	32.7	4.
4.25	21.78	1.742	1.32	28.8	26.03	1.957	1.39	36.2	4.25
4.5	23.62	1.809	1.35	31.8	28.12	2.000	1.41	39.8	4.5
5.	27.50	1.939	1.39	38.3	32.50	2.141	1.46	47.6	5.

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet	Bed 5 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 6 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet
0.5	2.625	.429	.65	1.72	3.125	.439	.66	2.07	0.5
0.75	4.031	.604	.77	3.14	4.781	.623	.79	3.79	0.75
1.	5.500	.760	.87	4.79	6.500	.789	.89	5.77	1.
1.25	7.031	.902	.95	6.68	8.281	.942	.97	8.04	1.25
1.5	8.625	1.032	1.02	8.76	10.12	1.082	1.04	10.53	1.5
1.75	10.28	1.156	1.08	11.04	12.03	1.214	1.10	13.31	1.75
2.	12.00	1.267	1.13	13.51	14.00	1.337	1.16	16.19	2.
2.25	13.78	1.374	1.17	16.16	16.03	1.453	1.21	19.33	2.25
2.5	15.625	1.476	1.21	18.98	18.125	1.564	1.25	22.67	2.5
2.75	17.531	1.572	1.25	21.98	20.281	1.669	1.29	26.21	2.75
3.	19.500	1.666	1.29	25.16	22.500	1.77[illegible]	1.33	29.94	3.
3.25	21.531	1.755	1.33	28.72	24.781	1.86[illegible]	1.37	33.97	3.25
3.5	23.625	1.854	1.36	32.[illegible]	27.125	1.964	1.40	37.99	3.5
3.75	25.781	1.928	1.39	35.78	29.531	2.052	1.43	42.31	3.75
4.	28.000	2.008	1.42	39.[illegible]	32.000	2.23[illegible]	1.46	46.83	4.
4.5	32.625	2.166	1.47	[illegible]	36.625	2.30[illegible]	1.52	55.65	4.5
5.	37.500	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	66.[illegible]	5.
6.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	90.[illegible]	6.

Depth in Feet	Bed 7 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 8 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 9 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 10 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	4.62	.457	.676	3.12	5.12	.461	.68	3.48	0.5
0.75	7.03	.659	.812	5.71	7.78	.666	.81	6.37	0.75
1.	9.5	.845	.919	8.73	10.5	.858	.93	9.73	1.
1.25	12.03	1.02	1.01	12.15	13.28	1.038	1.02	13.54	1.25
1.5	14.62	1.184	1.09	15.91	16.12	1.192	1.1	17.72	1.5
1.75	17.35	1.344	1.16	20.	19.03	1.367	1.17	22.26	1.75
2.	20.	1.485	1.22	24.37	22.	1.52	1.23	27.13	2.
2.25	22.78	1.624	1.28	29.03	25.03	1.665	1.29	32.31	2.25
2.5	25.62	1.756	1.33	33.96	28.12	1.804	1.34	37.78	2.5
2.75	28.53	1.883	1.38	39.15	31.28	1.937	1.39	43.54	2.75
3.	31.5	2.005	1.42	44.61	34.5	2.065	1.44	49.57	3.
3.25	34.53	2.121	1.46	50.31	37.78	2.188	1.48	55.88	3.25
3.5	37.62	2.236	1.5	56.26	41.12	2.308	1.52	62.46	3.5
3.75	40.78	2.346	1.54	62.47	44.53	2.422	1.56	69.31	3.75
4.	44.	2.446	1.57	68.91	48.	2.534	1.59	76.41	4.
4.25	47.28	2.555	1.6	75.59	51.53	2.642	1.63	83.77	4.25
4.5	50.62	2.656	1.63	82.51	55.12	2.748	1.66	91.38	4.5
5.	57.5	2.849	1.69	97.06	62.5	3.097	1.72	107.36	5.
6.	72.	3.212	1.79	129.03	78.	3.48	1.82	142.35	6.

Depth in Feet.	Bed 11 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 12 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	5.625	.464	.68	3.8	6.12	.467	.68	4.2	0.5
0.75	8.531	.673	.81	7.	9.28	.679	.82	7.6	0.75
1.	11.5	.869	.93	10.7	12.5	.878	.94	11.7	1.
1.25	14.531	1.053	1.02	14.9	15.78	1.067	1.03	16.3	1.25
1.5	17.625	1.228	1.11	19.5	19.1	1.244	1.12	21.3	1.5
1.75	20.78	1.393	1.18	24.5	22.5	1.414	1.19	26.8	1.75
2.	24.	1.551	1.25	29.9	26.	1.578	1.26	32.7	2.
2.25	27.281	1.702	1.31	35.6	29.5	1.732	1.32	38.9	2.25
2.5	30.625	1.846	1.36	41.6	33.1	1.882	1.37	45.5	2.5
2.75	34.031	1.984	1.41	47.9	36.8	2.028	1.42	52.4	2.75
3.	37.5	2.118	1.46	54.6	40.5	2.165	1.47	59.6	3.
3.25	41.03	2.246	1.5	61.5	44.3	2.299	1.52	67.1	3.25
3.5	44.63	2.372	1.54	68.7	48.1	2.427	1.56	75.	3.5
3.75	48.3	2.492	1.58	76.2	52.	2.551	1.6	83.1	3.75
4.	52.	2.607	1.61	84.	56.	2.674	1.64	91.6	4.
4.5	59.6	2.83	1.68	100.3	64.1	2.905	1.7	109.3	4.5
5.	67.5	2.021	1.74	117.8	72.5	3.128	1.77	128.2	5.
5.5	75.6	3.245	1.8	136.2	81.1	3.338	1.83	148.2	5.5
6.	84.	3.444	1.85	155.8	90.	3.541	1.88	169.4	6.

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 13 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 14 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	6.62	.469	.68	4.5	7.12	.471	.69	4.9	0.5
0.75	10.	6.682	.82	8.5	10.8	.689	.83	8.9	0.75
1.	13.5	.886	.94	12.	14.5	.893	.94	13.7	1.
1.25	17.	1.076	1.03	17.7	18.3	1.09	1.05	19.1	1.25
1.5	20.6	1.26	1.12	23.2	22.1	1.273	1.13	25.0	1.5
1.75	24.3	1.437	1.2	29.2	26.	1.451	1.2	31.4	1.75
2.	28.	1.603	1.27	35.4	30.	1.624	1.27	38.2	2.
2.25	31.8	1.764	1.33	42.2	34.	1.787	1.33	45.5	2.25
2.5	35.6	1.915	1.38	49.3	38.1	1.945	1.39	53.2	2.5
2.75	39.5	2.063	1.44	56.8	42.3	2.099	1.45	61.2	2.75
3.	43.5	2.207	1.49	64.6	46.5	2.246	1.5	69.7	3.
3.25	47.5	2.344	1.53	72.8	50.8	2.388	1.55	78.5	3.25
3.5	51.6	2.479	1.57	81.3	55.1	2.526	1.59	87.6	3.5
3.75	55.8	2.609	1.61	90.1	59.5	2.658	1.63	97.1	3.75
4.	60.	2.734	1.65	99.2	64.	2.79	1.67	106.	4.
4.5	68.6	2.975	1.73	118.4	73.1	3.038	1.74	127.5	4.5
5.	77.5	3.189	1.79	138.7	82.5	3.276	1.81	149.3	5.
5.5	86.6	3.423	1.85	160.3	92.1	3.502	1.87	172.4	5.5
6.	96.	3.634	1.91	183.	102.	3.72	1.93	196.7	6.

Depth in Feet.	Bed 15 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 16 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	7.62	.473	.69	5.2	8.12	.474	.69	5.6	0.5
0.75	11.5	.689	.83	9.6	12.3	.696	.83	10.2	0.75
1.	15.5	.899	.95	14.7	16.5	.905	.95	15.7	1.
1.25	19.5	1.096	1.05	20.5	20.8	1.161	1.05	21.8	1.25
1.5	23.6	1.289	1.13	26.8	25.1	1.297	1.14	28.6	1.5
1.75	27.8	1.47	1.21	33.7	29.5	1.482	1.22	37.	1.75
2.	32.	1.643	1.28	41.	34.	1.661	1.29	43.8	2.
2.25	36.3	1.812	1.34	49.2	38.5	1.831	1.35	52.6	2.25
2.5	40.6	1.972	1.4	57.1	43.1	1.996	1.41	60.9	2.5
2.75	45.	2.128	1.46	65.7	47.8	2.158	1.47	70.2	2.75
3.	49.5	2.28	1.51	74.7	52.5	2.312	1.52	79.8	3.
3.25	54.	2.425	1.56	84.2	57.3	2.463	1.57	89.9	3.25
3.5	58.6	2.568	1.6	93.9	62.1	2.608	1.61	100.3	3.5
3.75	63.3	2.707	1.65	104.1	67.	2.748	1.66	111.1	3.75
4.	68.	2.84	1.69	114.6	72.	2.887	1.7	122.3	4.
4.5	77.6	3.096	1.76	136.3	82.1	3.15	1.78	145.8	4.5
5.	87.5	3.342	1.83	160.	92.5	3.403	1.84	170.6	5.
5.5	97.6	3.575	1.89	184.6	103.1	3.643	1.91	196.9	5.5
6	108.	3.801	1.95	210.5	114.	3.875	1.97	224.4	6.

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 17 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 18 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.75	13.031	.696	.84	10.9	13.8	.701	.84	11.5	0.75
1.	17.5	.915	.95	16.7	18.5	.914	.96	17.7	1.
1.25	22.031	1.113	1.05	23.2	23.3	1.125	1.06	24.6	1.25
1.5	26.625	1.308	1.14	30.4	28.1	1.316	1.15	32.3	1.5
1.75	31.281	1.496	1.22	38.3	33.	1.506	1.23	40.6	1.75
2.	36.	1.677	1.29	46.6	38.	1.691	1.3	49.4	2.
2.25	40.8	1.852	1.36	55.5	43.	1.867	1.37	58.8	2.25
2.5	45.6	2.019	1.42	64.8	48.1	2.039	1.43	68.7	2.5
2.75	50.5	2.182	1.48	74.7	53.3	2.207	1.49	79.1	2.75
3.	55.5	2.341	1.53	84.9	58.5	2.368	1.54	90.	3.
3.25	60.5	2.493	1.58	95.6	63.8	2.525	1.59	101.3	3.25
3.5	65.6	2.643	1.63	106.7	69.1	2.677	1.64	113.1	3.5
3.75	70.8	2.789	1.67	118.2	74.5	2.824	1.68	125.3	3.75
4.	76.	2.93	1.71	130.1	80.	2.969	1.72	137.9	4.
4.5	86.6	3.2	1.79	155.	91.1	3.246	1.80	164.2	4.5
5.	97.5	3.46	1.86	181.4	102.5	3.513	1.87	192.1	5.
5.5	108.6	3.707	1.93	209.2	114.1	3.766	1.94	221.5	5.5
6.	120.	3.945	1.99	238.3	126.	4.014	2.	252.3	6.
7.	143.5	4.395	2.09	300.	150.5	4.472	2.11	318.3	7.

Depth in Feet.	Bed 19 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 20 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	9.62	.478	.69	6.7	10.1	.478	.69	7	0.5
0.75	14.5	.701	.84	12.2	15.3	.706	.84	13	0.75
1.	19.5	.918	.96	18.7	20.5	.922	.96	20	1.
1.25	24.5	1.124	1.06	26.	25.8	1.132	1.06	27	1.25
1.5	29.6	1.324	1.15	34.1	31.1	1.332	1.15	36	1.5
1.75	34.8	1.519	1.23	42.9	36.5	1.527	1.23	45	1.75
2.	40.	1.704	1.31	52.2	42.	1.716	1.31	55	2.
2.25	45.3	1.885	1.37	62.2	47.5	1.898	1.38	66	2.25
2.5	50.62	2.059	1.43	72.6	52.6	2.056	1.44	77	2.5
2.75	56.	2.227	1.49	83.6	58.8	2.249	1.5	88	2.75
3.	61.5	2.392	1.55	95.1	64.5	2.415	1.55	100	3.
3.25	67.	2.551	1.6	107.1	70.3	2.578	1.6	113	3.25
3.5	72.6	2.707	1.65	119.5	76.1	2.736	1.65	126	3.5
3.75	78.3	2.859	1.7	132.4	82.	2.889	1.7	139	3.75
4.	84.	3.006	1.74	145.6	88.	3.04	1.74	153	4.
4.25	89.8	3.151	1.78	159.3	94.	3.186	1.79	168	4.25
4.5	95.6	3.29	1.81	173.5	100.1	3.36	1.83	183	4.5
5.	107.5	3.629	1.89	202.9	112.5	3.608	1.9	214	5.
5.5	119.6	3.963	1.96	233.9	125.1	3.873	1.97	246	5.5
6.	132.	4.294	2.02	266.4	138.	4.13	2.03	280	6.
7.	157.5	4.545	2.13	335.8	164.5	4.614	2.15	353	7.
8.	184.	4.988	2.23	410.9	192.	5.068	2.25	432	8.

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 25 Feet.				Bed 30 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	12.12	.464	.7	9	15.1	.485	.7	11	0.5
0.75	19.03	.713	.85	16	22.8	.72	.85	19	0.75
1.	25.5	.936	.97	25	30.5	.946	.97	30	1.
1.25	32.03	1.152	1.08	34	38.3	1.168	1.06	41	1.25
1.5	38.62	1.362	1.17	45	46.1	1.382	1.18	54	1.5
1.75	45.28	1.566	1.25	57	54.	1.592	1.26	68	1.75
2.	52.	1.764	1.33	69	62.	1.798	1.34	83	2.
2.25	58.78	1.957	1.4	82	70.	1.996	1.41	91	2.25
2.5	65.62	2.145	1.46	96	78.1	2.194	1.48	116	2.5
2.75	72.53	2.329	1.52	111	86.3	2.387	1.54	133	2.75
3.	79.5	2.507	1.58	126	94.5	2.574	1.6	152	3.
3.25	86.53	2.681	1.64	142	102.8	2.758	1.66	171	3.25
3.5	93.62	2.853	1.69	158	111.1	2.938	1.71	190	3.5
3.75	100.78	3.019	1.74	175	119.5	3.113	1.76	211	3.75
4.	108.	3.182	1.78	193	128	3.287	1.81	232	4.
4.25	115.28	3.341	1.83	211	136.5	3.455	1.86	254	4.25
4.5	122.62	3.497	1.87	229	145.1	3.622	1.9	276	4.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	299	4.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	323	5.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.03	347	5.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.06	372	5.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	397	5.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	423	6
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	449	6.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	476	6.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	504	6.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	7.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	560	7.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	7.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	7.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	8
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	8.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	8.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	8.75
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	9

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 35 Feet.				Bed 40 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.75	26.53	.723	.85	23	40.37	.727	.85	26	0.75
1.	35.25	.947	.98	35	40.50	.959	.98	40	1.
1.25	44.53	1.178	1.09	48	50.78	1.187	1.09	55	1.25
1.5	53.62	1.398	1.18	63	61.02	1.408	1.19	73	1.5
1.75	62.78	1.613	1.27	80	71.58	1.630	1.28	91	1.75
2.	72.00	1.824	1.35	97	82.	1.844	1.36	111	2.
2.25	81.28	2.030	1.42	116	92.53	2.055	1.43	133	2.25
2.5	90.62	2.233	1.49	135	103.2	2.264	1.50	155	2.5
2.75	100.03	2.431	1.56	156	113.8	2.466	1.57	179	2.75
3.	109.50	2.625	1.62	177	124.5	2.665	1.63	203	3.
3.25	119.03	2.816	1.68	200	135.3	2.862	1.69	229	3.25
3.5	128.62	3.004	1.73	223	146.1	3.055	1.75	255	3.5
3.75	138.28	3.187	1.79	247	157.	3.245	1.80	283	3.75
4.	148.00	3.368	1.84	272	168.	3.433	1.85	311	4.
4.25	157.78	3.545	1.89	297	179.	3.617	1.90	340	4.25
4.5	167.62	3.720	1.93	323	190.1	3.797	1.95	371	4.5
4.75	177.53	3.891	1.97	350	201.3	3.977	2.00	401	4.75
5.	187.50	4.060	2.01	378	212.5	4.152	2.04	433	5.
5.25	197.53	4.226	2.05	406	223.8	4.326	2.08	465	5.25
5.5	207.62	4.390	2.10	435	235.1	4.495	2.12	499	5.5
5.75	217.78	4.551	2.14	465	246.5	4.664	2.16	532	5.75
6.	228.00	4.709	2.17	495	258.0	4.826	2.20	567	6.
6.25	238.28	4.865	2.21	526	269.5	4.993	2.24	602	6.25
6.5	248.62	5.019	2.24	557	281.1	5.155	2.27	638	6.5
6.75	259.03	5.171	2.28	589	292.8	5.315	2.31	675	6.75
7.	269.50	5.321	2.31	622	304.5	5.472	2.34	712	7.
7.25	280.03	5.468	2.34	655	316.3	5.627	2.37	750	7.25
7.5	290.62	5.614	2.37	689	328.1	5.779	2.40	789	7.5
7.75	301.28	5.756	2.40	723	340.	5.931	2.44	828	7.75
8.	312.00	5.900	2.43	758	352.	6.081	2.47	868	8.
8.25	322.78	6.039	2.46	793	364.	6.228	2.50	908	8.25
8.5	333.62	6.177	2.49	829	376.1	6.376	2.52	950	8.5
8.75	344.53	6.314	2.52	866	388.3	6.519	2.55	991	8.75
9.	355.50	6.449	2.54	903	400.5	6.661	2.58	1034	9.
9.5	377.62	6.714	2.59	979	425.1	6.941	2.63	1120	9.5
10.	400.00	6.974	2.64	1056	450.	7.216	2.69	1209	10.

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet	Bed 45 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 50 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.50
0.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	30.5	1.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.50
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 9.

Channels having a trapezoidal section, with side slopes of ½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

BED 60 FEET.

Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$
1.	60.50	.972	.99	60
1.5	91.12	1.438	1.20	109
1.75	106.53	1.667	1.29	137
2.	122.	1.892	1.38	168
2.25	137.53	2.115	1.46	200
2.5	153.12	2.334	1.53	234
2.75	168.78	2.552	1.60	270
3.	184.50	2.781	1.66	307
3.25	200.28	2.977	1.73	346
3.5	216.12	3.188	1.79	386
3.75	232.03	3.378	1.84	427
4.	248.	3.597	1.90	470
4.25	264.03	3.799	1.96	515
4.5	280.12	3.998	2.	560
4.75	296.28	4.195	2.05	607
5.	312.50	4.390	2.10	655
5.25	328.78	4.583	2.15	704
5.5	345.12	4.774	2.18	754
5.75	361.53	4.962	2.23	805
6.	378.	5.149	2.27	858
6.25	394.53	5.333	2.31	911
6.5	411.12	5.516	2.35	965
6.75	427.78	5.697	2.39	1021
7.	444.50	5.876	2.42	1077
7.25	461.28	6.053	2.46	1135
7.5	478.12	6.228	2.50	1193
7.75	495.03	6.363	2.53	1252
8.	512.	6.574	2.56	1313
8.25	529.03	6.744	2.59	1374
8.5	546.12	9.912	2.63	1436
8.75	563.28	7.079	2.66	1499
9.	580.50	7.245	2.69	1563
9.5	615.12	7.571	2.75	1693
10.	650.	7.892	2.81	1826
10.5	685.12	8.271	2.86	1963
11.	720.50	8.517	2.92	2103

6

TABLE 10.

Sectional areas, in square feet, of trapezoidal channels, with side slopes of ½ to 1.

Depth in Feet.	Bed Width				
	70 feet	80 feet.	90 feet.	100 feet.	120 feet.
1.	70.50	80.50	90.50	100.50	120.50
1.5	106.12	121.12	136.12	151.12	181.12
2.	142.	162.	182.	202.	242.
2.25	160.03	182.53	205.03	227.53	272.53
2.5	178.12	203.12	228.12	253.12	303.12
2.75	196.28	224.78	252.28	278.78	333.78
3	214.50	244.50	274.50	304.50	364.50
3.25	232.78	265.28	297.78	330.28	395.28
3.5	251.12	286.12	321.12	356.12	426.12
3.75	269.53	307.03	344.53	382.03	457.03
4	288.	328.	368.	408.	488.
4.25	306.53	349.03	391.53	434.03	519.03
4.5	325.12	370.12	415.12	460.12	550.12
4.75	343.78	391.28	438.78	486.28	581.28
5.	362.50	412.50	462.50	512.50	612.50
5.25	381.28	433.78	486.28	538.78	663.78
5.5	400.12	455.12	510.12	565.12	675.12
5.75	419.03	476.53	534.03	591.53	706.53
6.	438.	498.	558.	618.	738.
6.25	457.03	519.53	582.03	644.53	769.53
[illegible]	[illegible]	[illegible]	606.12	[illegible]	801.12
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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TABLE 10.

Sectional areas, in square feet, of trapezoidal channels, with side slopes of ½ to 1.

Depth in Feet.	Bed Width				
	140 feet.	160 feet.	180 feet.	200 feet.	220 feet.
1.	140.50	160.50	180.50	200.50	220.50
2.	282.	322.	362.	402.	442.
2.5	353.12	403.12	453.12	503.10	553.12
2.75	388.78	443.78	498.78	553.78	608.78
3.	424.50	484.50	544.50	604.50	664.50
3.25	460.28	525.28	590.28	655.28	720.28
3.5	496.12	566.12	636.12	706.12	776.12
3.75	532.03	607.03	682.03	757.03	832.03
4.	568.	648.	728.	808.	888.80
4.25	604.03	689.03	774.03	859.03	944.03
4.5	640.12	730.12	820.12	910.12	1000.12
4.75	676.28	771.28	866.28	961.28	1056.28
5.	712.50	812.50	912.50	1012.50	1112.50
5.25	748.78	853.78	958.78	1063.78	1168.78
5.5	785.12	895.12	1005.12	1115.12	1225.12
5.75	821.53	936.53	1051.53	1166.53	1281.53
6.	858.	978.	1098.	1218.	1338.
6.25	894.53	1019.53	1144.53	1269.53	1394.53
6.5	931.12	1061.12	1191.12	1321.12	1451.12
6.75	967.78	1102.78	1237.78	1372.78	1507.78
7.	1004.50	1144.50	1284.50	1424.50	1564.50
7.25	1041.28	1186.28	1331.28	1476.28	1621.28
7.5	1078.12	1228.12	1378.12	1528.12	1678.12
7.75	1115.03	1270.03	1425.03	1580.03	1735.03
8.	1152.	1312.	1472.	1632.	1792.
8.25	1189.03	1354.03	1519.03	1684.03	1849.03
8.5	1226.12	1396.12	1566.12	1736.12	1906.12
8.75	1263.28	1438.28	1613.28	1788.28	1963.28
9.	1300.50	1480.50	1660.50	1840.50	2020.50
9.25	1337.78	1522.78	1707.78	1892.78	2077.78
9.5	1375.12	1565.12	1755.12	1945.12	2135.12
9.75	1412.53	1607.53	1802.53	1997.53	2192.53
10.	1450.	1650.	1850.	2050.	2250.
10.5	1525.12	1735.12	1945.12	2155.12	2365.12
11.	1600.50	1820.50	2040.50	2260.50	2480.50
11.5	1676.12	1906.12	2136.12	2366.12	2596.12
12.	1752.	1992.	2232.	2472.	2712.
13.	1904.50	2164.50	2424.50	2684.50	2944.50
14.	2058.	2338.	2618.	2898.	3178.
15.	2212.50	2512.50	2812.50	3112.50	3412.50
16.	2368.	2688.	3008.	3328.	3648.

TABLE 10.

Sectional areas, in square feet, of trapezoidal channels, with side slopes of ½ to 1.

Depth in Feet.	Bed Width			
	240 feet.	260 feet.	280 feet.	300 feet.
1.	240.50	260.50	280.50	300.50
2.	482.	522.	562.	602.
2.5	603.12	653.12	703.12	753.12
2.75	663.78	718.78	773.78	828.78
3.	724.50	784.50	844.50	904.50
3.25	785.28	850.28	915.28	980.28
3.5	846.12	916.12	986.12	1056.12
3.75	907.03	982.03	1057.03	1132.03
4.	968.	1048.	1128.	1208.
4.25	1029.03	1114.03	1199.03	1284.03
4.5	1090.12	1180.12	1270.12	1360.12
4.75	1151.28	1246.28	1341.28	1436.28
5.	1212.50	1312.50	1412.50	1512.50
5.25	1273.78	1378.78	1483.78	1588.78
5.5	1335.12	1445.12	1555.12	1665.12
5.75	1396.53	1511.53	1626.53	1741.53
6.	1458.	1578.	1698.	1818.
6.25	1519.53	1644.53	1769.53	1894.53
6.5	1581.12	1711.12	1841.12	1971.12
6.75	1642.78	1777.78	1912.78	2047.78
7.	1704.50	1844.50	1984.50	2124.50
7.25	1766.28	1911.28	2056.28	2201.28
7.5	1828.12	1978.12	2128.12	2278.12
7.75	1890.03	2045.03	2200.03	2355.03
8.	1952.	2112.	2272.	2432.
8.25	2014.03	2179.03	2344.03	2509.03
8.5	2076.12	2246.12	2416.12	2586.12
8.75	2138.28	2313.28	2488.28	2663.28
9.	2200.50	2380.50	2560.50	2740.50
9.25	2262.78	2447.78	2632.78	2817.78
9.5	2325.12	2515.12	2705.12	2895.12
9.75	2387.53	2582.53	2777.53	2972.53
10.	2450.	2650.	2850.	3050.
10.5	2575.12	2785.12	2995.12	3205.12
11.	2700.50	2920.50	3140.50	3360.50
11.5	2826.12	3156.12	3486.12	3816.12
12.	2952.	3192.	3432.	3672.
13.	3204.50	3464.50	3724.50	3984.50
14.	3458.	3738.	4018.	4298.
15.	3712.50	4012.50	4312.50	4412.50
16.	3968.	4288.	4608.	4928.

TABLE 11.

Channels having a trapezoidal section, with side slopes of $1\frac{1}{2}$ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and also } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 1 Foot.				Bed 2 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	.87	.312	.56	.49	1.375	.362	.60	.83	0.5
0.75	1.59	.452	.65	1.04	2.344	.499	.71	1.66	0.75
1.	2.5	.542	.74	1.84	3.5	.624	.79	2.76	1.
1.25	3.59	.652	.81	2.89	4.844	.744	.86	4.17	1.25
1.5	4.87	.761	.87	4.24	6.37	.860	.93	5.93	1.5
1.75	6.34	.868	.93	5.9	8.09	.974	.99	8.	1.75
2.	8.	.974	.99	7.9	10.	1.086	1.04	10.4	2.
2.25	9.84	1.081	1.04	10.2	12.09	1.196	1.09	13.2	2.25
2.5	11.87	1.186	1.09	12.9	14.37	1.294	1.14	16.4	2.5
2.75	14.09	1.280	1.14	16.1	16.84	1.414	1.19	20.	2.75
3.	16.5	1.397	1.18	19.5	19.50	1.521	1.23	24.	3.
3.25					22.34	1.629	1.28	28.5	3.25
3.5					25.37	1.736	1.32	33.4	3.5
3.75					28.6	1.842	1.36	38.8	3.75
4.					32.	1.949	1.39	44.4	4.

	Bed 3 Feet.				Bed 4 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	1.875	.499	.63	1.17	2.37	.409	.64	1.51	0.5
0.75	3.094	.543	.73	2.29	3.84	.574	.76	2.92	0.75
1.	4.50	.681	.83	3.71	5.5	.723	.85	4.67	1.
1.25	6.09	.811	.90	5.48	7.34	.863	.93	6.83	1.25
1.5	7.87	.935	.97	7.62	9.37	.996	1.	9.38	1.5
1.75	9.84	1.057	1.03	10.1	11.59	1.125	1.06	12.3	1.75
2.	12.	1.175	1.08	13.	14.	1.248	1.12	15.7	2.
2.25	14.34	1.291	1.14	16.4	16.59	1.370	1.17	19.4	2.25
2.5	16.87	1.405	1.19	20.1	19.37	1.489	1.22	23.6	2.5
2.75	19.59	1.518	1.23	24.1	22.34	1.607	1.27	28.4	2.75
3.	22.50	1.628	1.28	28.8	25.50	1.721	1.31	33.4	3.
3.25	25.60	1.739	1.32	33.8	28.84	1.835	1.36	39.2	3.25
3.5	28.87	1.848	1.36	39.3	32.37	1.947	1.40	45.3	3.5
3.75	32.34	1.958	1.40	45.3	36.09	2.060	1.44	52.	3.75
4.	36.	2.067	1.44	51.8	40.	2.171	1.47	59.	4.
4.25	39.84	2.175	1.48	59.	44.09	2.282	1.51	66.6	4.25
4.5	43.87	2.283	1.51	66.3	48.37	2.392	1.55	75.	4.5
5.	52.5	2.497	1.58	83.	57.50	2.610	1.62	92.9	5.

TABLE 11.

Channels having a trapezoidal section, with side slopes of 1½ to 1. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 5 Feet.				Bed 6 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	2.875	.423	.64	1.87	3.37	.433	.66	2.23	0.5
0.75	4.59	.597	.77	3.54	5.34	.614	.78	4.17	0.75
1.	6.5	.755	.87	5.64	7.5	.780	.89	6.62	1.
1.25	8.59	.904	.95	8.17	9.84	.937	.97	9.55	1.25
1.5	10.87	1.045	1.02	11.09	12.37	1.084	1.04	12.9	1.5
1.75	13.34	1.179	1.09	14.54	15.09	1.226	1.11	16.8	1.75
2.	16.	1.310	1.15	18.24	18.	1.362	1.17	21.	2.
2.25	18.84	1.437	1.20	22.61	21.09	1.495	1.23	26.	2.25
2.5	21.87	1.560	1.25	27.33	24.37	1.623	1.28	31.2	2.5
2.75	25.09	1.683	1.30	32.62	27.84	1.750	1.33	37.	2.75
3.	28.5	1.802	1.34	38.20	31.5	1.873	1.37	43.2	3.
3.25	32.09	1.919	1.39	44.61	35.34	1.995	1.41	49.8	3.25
3.5	35.87	2.036	1.43	51.30	39.37	2.114	1.45	57.1	3.5
3.75	39.84	2.153	1.47	58.57	43.59	2.233	1.49	65.	3.75
4.	44.	2.266	1.51	66.40	48.	2.350	1.53	73.6	4.
4.5	52.87	2.491	1.58	83.54	57.37	2.581	1.60	91.8	4.5
5.	62.50	2.713	1.64	103.	67.50	2.808	1.67	113.1	5.
6.	84.	3.153	1.78	149.5	90.	3.256	1.81	162.9	6.

	Bed 7 Feet.				Bed 8 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	3.87	.440	.67	2.57	4.37	.446	.67	2.92	0.5
0.75	6.09	.623	.79	4.81	6.84	.640	.80	5.48	0.75
1.	8.5	.801	.89	7.61	9.5	.818	.90	8.58	1.
1.25	11.09	.965	.98	10.87	12.34	.987	.99	12.2	1.25
1.5	13.87	1.119	1.06	14.71	15.37	1.146	1.07	16.5	1.5
1.75	16.84	1.266	1.12	18.90	18.59	1.299	1.14	21.2	1.75
2.	20.	1.407	1.18	23.70	22.	1.446	1.20	26.5	2.
2.25	23.34	1.545	1.24	29.	25.59	1.589	1.26	32.3	2.25
2.5	26.87	1.679	1.30	34.9	29.37	1.726	1.31	38.5	2.5
2.75	30.59	1.809	1.35	41.3	33.34	1.862	1.36	45.4	2.75
3.	34.50	1.936	1.39	48.	37.50	1.993	1.41	52.9	3.
3.25	38.59	2.062	1.44	55.6	41.84	2.125	1.46	61.1	3.25
3.5	42.87	2.184	1.48	63.4	46.37	2.248	1.50	69.6	3.5
3.75	47.34	2.307	1.52	72.	51.09	2.374	1.54	78.7	3.75
4.	52.	2.427	1.56	81.1	56.	2.497	1.58	88.5	4.
4.5	61.87	2.664	1.63	100.9	66.37	2.739	1.65	109.5	4.5
5.	72.50	2.897	1.70	123.3	77.50	2.976	1.72	133.3	5.
6.	96.	3.353	1.83	175.8	102.	3.442	1.85	189.2	6.

TABLE 11.

Channels having a trapezoidal section, with side slopes of $1\frac{1}{2}$ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 9 Feet.				Bed 10 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	4.875	.451	.68	3.28	5.375	.456	.68	3.63	0.5
0.75	7.59	.649	.81	6.15	8.344	.657	.81	6.15	0.75
1.	10.5	.833	.91	9.58	11.5	.845	.92	10.58	1.
1.25	13.594	1.006	1.	13.6	14.844	1.023	1.01	15.	1.25
1.5	16.875	1.170	1.08	18.3	18.375	1.192	1.09	20.	1.5
1.75	20.344	1.329	1.15	23.4	22.094	1.355	1.16	25.6	1.75
2.	24.	1.480	1.22	29.3	26.	1.510	1.23	32.	2.
2.25	27.844	1.623	1.28	35.5	30.094	1.662	1.29	38.8	2.25
2.5	31.875	1.769	1.33	42.4	34.375	1.807	1.34	46.2	2.5
2.75	36.094	1.909	1.38	49.8	38.844	1.951	1.39	54.	2.75
3.	40.5	2.044	1.43	57.9	43.5	2.090	1.44	62.6	3.
3.25	45.094	2.176	1.48	66.7	48.344	2.223	1.49	72.	3.25
3.5	49.875	2.306	1.52	75.8	53.375	2.358	1.54	82.2	3.5
3.75	54.844	2.440	1.56	85.6	58.594	2.491	1.58	92.6	3.75
4.	60.	2.561	1.60	96.	64.	2.620	1.62	103.6	4.
4.25	65.344	2.687	1.64	107.2	69.594	2.749	1.66	115.5	4.25
4.5	70.875	2.810	1.68	118.8	75.375	2.873	1.70	128.1	4.5
5.	82.5	3.052	1.75	144.4	87.5	3.121	1.77	154.6	5.
6.	108.	3.525	1.877	202.7	114.	3.604	1.9	216.6	6.

	Bed 11 Feet.				Bed 12 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	5.87	.459	.68	3.99	6.37	.462	.68	4.33	0.5
0.75	9.094	.664	.81	7.37	9.844	.670	.82	8.07	0.75
1.	12.5	.856	.93	11.63	13.5	.865	.93	12.55	1.
1.25	16.094	1.038	1.02	16.42	17.344	1.051	1.02	17.7	1.25
1.5	19.875	1.211	1.10	21.86	21.375	1.228	1.11	23.7	1.5
1.75	23.844	1.377	1.17	27.90	25.594	1.398	1.18	30.2	1.75
2.	28.	1.537	1.24	34.7	30.	1.561	1.25	37.5	2.
2.25	32.344	1.693	1.30	42.	34.594	1.720	1.31	45.3	2.25
2.5	36.875	1.842	1.36	50.2	39.375	1.874	1.37	53.9	2.5
2.75	41.594	1.989	1.41	58.6	44.344	2.024	1.42	63.	2.75
3.	46.5	2.132	1.46	67.9	49.5	2.170	1.47	72.9	3.
3.25	51.594	2.271	1.51	77.9	54.844	2.312	1.52	83.4	3.25
3.5	56.875	2.407	1.55	88.2	60.375	2.452	1.57	94.8	3.5
3.75	62.344	2.543	1.59	99.1	66.094	2.590	1.61	106.4	3.75
4.	68.	2.675	1.64	111.5	72.	2.725	1.65	118.9	4.
4.5	79.875	2.933	1.71	136.6	84.375	2.990	1.73	146.	4.5
5.	92.5	3.186	1.78	164.6	97.5	3.247	1.80	175.5	5.
5.5	105.875	3.434	1.85	196.2	111.375	3.499	1.87	208.3	5.5
6.	120.	3.676	1.92	230.4	126.	3.746	1.94	244.	6.

TABLE 11.

Channels having a trapezoidal section, with side slopes of 1½ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formula

$$v = c\sqrt{r}\sqrt{s} \text{ and } Q = ac\sqrt{r}\sqrt{s}$$

Depth in Feet	Bed 13 Feet				Bed 14 Feet				Depth in Feet
	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Depth in Feet									Depth in Feet
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 11.

Channels having a trapezodial section, with side slopes of 1½ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and also } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 17 Feet.				Bed 18 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.75	13.59	.690	.83	11.3	14.34	.693	.83	11.9	0.75
1.	18.50	.897	.95	17.6	19.5	.902	.95	18.5	1.
1.25	23.59	1.097	1.05	24.8	24.84	1.104	1.05	26.1	1.25
1.5	28.87	1.288	1.13	32.6	30.37	1.297	1.14	34.6	1.5
1.75	34.34	1.473	1.21	41.6	36.09	1.485	1.22	44.	1.75
2.	40.	1.652	1.29	51.6	42.	1.665	1.29	54.2	2.
2.25	45.84	1.810	1.35	61.9	48.09	1.842	1.36	65.3	2.25
2.5	51.87	1.993	1.41	73.1	54.37	2.013	1.42	77.2	2.5
2.75	58.09	2.159	1.47	85.4	60.84	2.180	1.48	90.	2.75
3.	64.50	2.318	1.52	98.	67.50	2.342	1.53	106.3	3.
3.25	71.09	2.475	1.57	111.6	74.34	2.501	1.58	117.5	3.25
3.5	77.87	2.628	1.62	126.2	81.37	2.658	1.63	132.6	3.5
3.75	84.84	2.780	1.67	141.7	88.59	2.811	1.68	148.8	3.75
4.	92.	2.927	1.71	157.3	96.	2.961	1.72	165.2	4.
4.5	106.87	3.216	1.79	191.	111.37	3.254	1.80	200.8	4.5
5.	122.50	3.496	1.87	229.	127.50	3.539	1.88	239.7	5.
5.5	138.87	3.771	1.94	269.	144.37	3.816	1.95	281.5	5.5
6.	156.	4.037	2.01	314.	182.	4.087	2.02	327.4	6.
7.	192.50	4.557	2.135	411.	199.50	4.614	2.15	428.9	7.

	Bed 19 Feet.				Bed 20 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.75	15.09	0.695	0.834	12.6	15.80	.698	.835	13.2	0.75
1.	20.5	0.906	0.952	20.5	21.50	.910	.95	20.4	1.
1.25	26.09	1.1	1.053	27.5	27.34	1.116	1.05	28.7	1.25
1.5	31.87	1.305	1.142	36.3	33.37	1.313	1.15	38.4	1.5
1.75	37.84	1.459	1.223	46.3	39.59	1.505	1.23	48.7	1.75
2.	44.	1.678	1.295	57.	46.	1.690	1.30	59.8	2.
2.25	50.34	1.857	1.363	68.6	52.59	1.871	1.37	72.1	2.25
2.5	56.87	2.03	1.425	81.	59.37	2.046	1.43	85.5	2.5
2.75	63.59	2.199	1.483	94.3	66.34	2.218	1.49	98.9	2.75
3.	70.5	2.364	1.538	108.4	73.50	2 386	1.54	113.2	3.
3.25	77.59	2.526	1.589	123.3	80.84	2.549	1.60	129.4	3.25
3.5	84.87	2.683	1.64	139.2	88.37	2.708	1.65	145.8	3.5
3.75	92.34	2.839	1.685	155.6	96.09	2.867	1.69	162.4	3.75
4.	100.	2.992	1.709	170.9	104.	3.021	1.73	179.9	4.
4.25	107.84	3.142	1.772	191.1	112.09	3.174	1.78	199.5	4.25
4.5	115.87	3.289	1.813	210.	120.37	3.322	1.82	219.1	4.5
5.	132.5	3.577	1.892	250.5	137.5	3.615	1.90	261.7	5.
5.5	149.87	3.855	1.964	294.3	155.37	3.901	1.97	306.	5.5
6.	168.	4.134	2.033	341.5	174.	4.179	2.04	355.	6.
7.	206.5	4.668	2.16	446.	213.5	4.719	2.17	463.7	7.
8.	248.	5.183	2.277	564.7	256.	5.241	2.28	583.7	8.

TABLE 11.

Channels having a trapezoidal section, with side slopes of 1½ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 25 Feet.				Bed 30 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	12.87	.480	.693	8.92	15.37	.483	.695	10.69	0.5
0.75	19.59	.707	.841	16.5	23.34	.714	.845	19.7	0.75
1.	26.50	.926	.962	25.5	39.06	.937	.968	37.8	1.
1.25	33.59	1.138	1.067	35.8	47.81	1.132	1.064	50.9	1.25
1.5	40.87	1.344	1.16	47.4	48.37	1.366	1.17	56.3	1.5
1.75	48.34	1.544	1.24	60.	57.09	1.572	1.25	71.4	1.75
2	56.	1.733	1.32	73.9	66.	1.774	1.33	87.8	2.
2.25	63.844	1.922	1.39	88.7	75.09	1.970	1.40	105.1	2.25
2.5	71.875	2.107	1.45	104.3	84.37	2.167	1.47	124.3	2.5
2.75	80.094	2.294	1.51	120.9	93.84	2.351	1.53	143.6	2.75
3	88.5	2.471	1.57	139.	103.59	2.536	1.59	165.2	3.
3.25	97.094	2.645	1.63	158.	113.34	2.717	1.65	187.	3.25
3.5	105.875	2.814	1.68	177.	123.37	2.895	1.70	209.7	3.5
3.75	114.844	2.982	1.73	199.	133.59	3.070	1.75	233.8	3.75
4	124.	3.146	1.78	221.	144.	3.242	1.80	259.2	4.
4.25	133.344	3.307	1.82	243.	154.59	3.411	1.85	286.	4.25
4.5	[illegible]	3.466	[illegible]	[illegible]	[illegible]	[illegible]	1.89	312.6	4.5
4.75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	340.3	4.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	5.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	6.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 11.

Channels having a trapezoidal section, with side slopes of 1½ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 35 Feet.				Bed 40 Feet.				Depth in Feet.
	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	
0.75	27.09	.719	.847	22.95	30.84	.722	.85	26.2	0.75
1.	36.50	.945	.972	35.5	41.5	.952	.976	40.5	1.
1.25	46.11	1.167	1.080	49.8	52.3	1.176	1.084	56.7	1.25
1.5	55.87	1.383	1.176	65.7	63.4	1.396	1.181	74.9	1.5
1.75	65.844	1.594	1.26	83.4	76.34	1.648	1.28	97.7	1.75
2.	76.	1.801	1.34	101.8	86.	1.822	1.35	115.	2.
2.25	86.344	2.003	1.41	121.7	97.59	2.029	1.42	138.6	2.25
2.5	96.875	2.201	1.48	143.2	109.37	2.232	1.49	163.	2.5
2.75	107.594	2.396	1.55	166.8	121.34	2.431	1.56	189.3	2.75
3.	118.5	2.587	1.61	190.8	133.50	2.627	1.62	216.3	3.
3.25	129.594	2.774	1.67	216.4	145.84	2.839	1.68	245.	3.25
3.5	140.875	2.958	1.72	242.4	158.37	3.010	1.73	274.	3.5
3.75	152.344	3.140	1.77	269.6	171.09	3.197	1.79	306.	3.75
4.	164.	3.318	1.82	298.5	184.	3.399	1.84	338.	4.
4.25	175.844	3.495	1.87	329.	197.09	3.563	1.89	373.	4.25
4.5	187.875	3.668	1.91	359.	210.37	3.742	1.93	406.	4.5
4.75	200.094	3.839	1.96	392.	223.84	3.919	1.98	443.	4.75
5.	212.5	4.007	2.	425.	237.50	4.094	2.03	481.	5.
5.25	225.094	4.174	2.04	459.	251.34	4.265	2.07	520.	5.25
5.5	237.875	4.338	2.08	495.	265.37	4.435	2.11	560.	5.5
5.75	250.8	4.501	2.12	535.3	279.6	4.604	2.15	601.	5.75
6.	264.	4.661	2.16	570.	294.	4.770	2.18	641.	6.
6.25	277.3	4.820	2.19	608.7	308.6	4.935	2.22	685.	6.25
6.5	290.9	4.977	2.23	649.	323.4	5.097	2.26	731.	6.5
6.75	304.6	5.133	2.26	689.9	338.3	5.259	2.29	776.	6.75
7.	318.5	5.287	2.30	732.2	353.5	5.418	2.33	823.	7.
7.25	332.6	5.440	2.33	775.6	368.8	5.577	2.36	871.	7.25
7.5	346.9	5.591	2.36	820.4	384.4	5.733	2.39	920.	7.5
7.75	351.3	5.741	2.39	841.7	400.1	5.889	2.43	970.	7.75
8.	376.	5.889	2.42	912.2	416.	6.043	2.46	1023.	8.
8.25	390.8	6.037	2.45	960.2	432.1	6.195	2.49	1075.	8.25
8.5	405.9	6.183	2.48	1009.	448.4	6.347	2.52	1130.	8.5
8.75	421.1	6.327	2.51	1059.	464.8	6.497	2.55	1185.	8.75
9.	436.5	6.471	2.54	1110.	481.5	6.646	2.58	1241.	9.
9.5	467.9	6.756	2.60	1216.	515.4	6.941	2.64	1358.	9.5
10.	500.	7.037	2.65	1327.	550.	7.232	2.69	1479.	10.

TABLE 11.

Channels having a trapezoidal section, with side slopes of 1½ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 45 Feet.				Bed 50 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	22.87	.490	.700	16.	25.37	.490	.700	17.8	0.5
0.75	34.59	.725	.852	29.5	38.34	.728	.85[illegible]	32.7	0.75
1.	46.50	.957	.977	45.4	51.50	.961	.980	50.5	1.
1.25	58.57	1.183	1.084	63.5	64.84	1.190	1.091	70.7	1.25
1.5	70.88	1.406	1.190	84.3	78.37	1.415	1.190	93.3	1.5
1.75	83.34	1.624	1.274	106.2	92.09	1.635	1.28	118.	1.75
2.	96.	1.839	1.356	130.2	106.	1.853	1.36	138.	2.
2.25	108.8	2.049	1.43	156.	120.09	2.067	1.44	173.	2.25
2.5	121.9	2.257	1.50	183.	134.37	2.277	1.51	202.	2.5
2.75	135.1	2.460	1.57	212.	148.84	2.484	1.58	235.	2.75
3.	148.5	2.660	1.63	242.	163.50	2.688	1.64	268.	3.
3.25	162.1	2.858	1.69	274.	178.34	2.890	1.70	303.	3.25
3.5	175.9	3.052	1.75	308.	193.37	3.088	1.76	340.	3.5
3.75	189.8	3.244	1.80	342.	208.59	3.284	1.81	378.	3.75
4.	204.	3.433	1.85	377.	224.	3.477	1.86	417.	4.
4.25	218.3	3.620	1.90	415.	239.59	3.668	1.92	460.	4.25
4.5	232.9	3.804	1.95	454.	255.37	3.856	1.96	501.	4.5
4.75	247.6	3.985	2.	495.	271.34	4.043	2.01	545.	4.75
5.	262.5	4.165	2.04	536.	287.50	4.226	2.05	591.	5.
5.25	277.6	4.342	2.08	577.	303.84	4.408	2.10	638.	5.25
5.5	292.88	4.518	2.13	624.	320.4	4.588	2.14	686.	5.5
5.75	308.34	4.683	2.16	667.2	337.1	4.766	2.18	735.	5.75
6.	324.	4.862	2.20	713.	354.	4.941	2.22	786.	6.
6.25	339.84	5.032	2.25	763.6	371.1	5.116	2.26	839.	6.25
6.5	355.99	5.200	2.28	811.	388.4	5.288	2.30	893.	6.5
6.75	372.1	5.366	2.31	861.8	405.8	5.461	2.33	948.	6.75
7.	388.5	5.531	2.35	913.	423.5	5.628	2.37	1005.	7.
7 25	405.7	5.703	2.39	968.8	441.3	5.799	2.40	1063.	7.25
7.5	421.9	5.856	2.42	1021.	459.4	5.963	2.44	1122.	7.5
7.75	438.8	6.016	2.45	1076.	477.6	6.128	2.47	1182.	7.75
8.	456.	6.175	2.48	1133.	496.	6.291	2.50	1244.	8.
8.25	473.3	6.333	2.51	1191.	514.6	6.453	2.54	1307.	8.25
8.5	490.9	6.489	2.55	1250.	533.4	6.613	2.57	1371.	8.5
8.75	508.6	6.644	2.58	1311.	552.3	6.773	2.60	1437.	8.75
9.	526.5	6.798	2.61	1373.	571.5	6.931	2.63	1503.	9.
9.5	562.9	7.102	2.66	1500.	610.4	7.244	2.69	1642.	9.5
10.	600.	7.414	2.72	1633.	650.	7.553	2.75	1786.	10.
10.5	637.9	7.869	2.78	1773.	690.4	7.858	2.80	1933.	10.5
11.	676.5	7.991	2.83	1912.	731.5	8.158	2.86	209.2	11.

TABLE 11.

Channels having a trapezoidal section, with side slopes of 1½ to 1. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 60 Feet.				Bed 70 Feet.				
Depth in feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in feet.
1.	61.50	.951	.978	59.2	71.5	.9713	.98	70.	1.
1.5	91.12	1.393	1.180	107.5	108.37	1.437	1.19	129.	1.5
1.75	109.59	1.647	1.29	141.4	127.09	1.666	1.29	164.	1.75
2.	126.	1.875	1.37	172.6	146.	1.891	1.37	200.	2.
2.25	142.60	2.094	1.45	206.8	165.09	2.114	1.45	239.	2.25
2.5	159.38	2.309	1.52	242.3	184.37	2.334	1.53	282.	2.5
2.75	176.34	2.522	1.59	280.4	203.84	2.551	1.60	326.	2.75
3.	193.50	2.732	1.65	320.	223.5	2.765	1.66	371.	3.
3.25	210.84	2.940	1.71	360.	243.34	2.978	1.73	421.	3.25
3.5	228.37	3.145	1.77	404.	263.37	3.188	1.79	471.	3.5
3.75	246.09	3.347	1.83	450.	283.59	3.396	1.84	522.	3.75
4.	264.	3.547	1.88	496.	304.	3.601	1.90	578.	4.
4.25	282.09	3.745	1.94	547.	324.59	3.804	1.95	633.	4.25
4.5	300.37	3.941	1.99	598.	345.38	4.006	2.	691.	4.5
4.75	318.84	4.134	2.03	647.	366.34	4.205	2.05	751.	4.75
5.	337.50	4.325	2.08	702.	387.5	4.402	2.10	814.	5.
5.25	356.34	4.515	2.12	755.	408.8	4.597	2.14	875.	5.25
5.5	375.37	4.702	2.17	815.	430.4	4.791	2.19	943.	5.5
5.75	394.59	4.888	2.21	872.	452.09	4.983	2.23	1008.	5.75
6.	414.	5.071	2.25	932.	474.	5.172	2.27	1076.	6.
6.25	433.59	5.253	2.29	993.	496.09	5.361	2.32	1151.	6.25
6.5	453.37	5.434	2.33	1056.	518.4	5.548	2.36	1223.	6.5
6.75	473.34	5.612	2.37	1122.	540.84	5.733	2.39	1293.	6.75
7.	493.50	5.789	2.40	1188.	563.5	5.916	2.43	1369.	7.
7.25	513.84	5.965	2.44	1255.	586.34	6.099	2.47	1448.	7.25
7.5	534.37	6.139	2.47	1325.	609.4	6.279	2.51	1527.	7.5
7.75	555.09	6.312	2.51	1394.	632.59	6.459	2.54	1607.	7.75
8.	576.	6.483	2.54	1466.	656.	6.636	2.57	1686.	8.
8.25	597.09	6.605	2.58	1546.	679.59	6.813	2.61	1774.	8.25
8.5	618.37	6.822	2.61	1615.	703.4	6.988	2.64	1859.	8.5
8.75	639.84	6.989	2.64	1690.	727.34	7.162	2.68	1949.	8.75
9.	661.50	7.155	2.67	1770.	751.5	7.335	2.71	2036.	9.
9.5	705.37	7.484	2.73	1929.	800.4	7.677	2.77	2218.	9.5
10.	750.	7.808	2.79	2096.	850.	8.014	2.83	2406.	10.
10.5	795.37	8.128	2.85	2268.	900.4	8.347	2.90	2601.	10.5
11.	841.5	8.444	2.90	2445.	951.5	8.676	2.94	2802.	11.

TABLE 12.

Sectional areas, in square feet, of trapezoidal channels, with side slopes of 1½ to 1.

Depth in Feet.	Bed Width				
	70 feet.	80 feet.	90 feet.	100 feet.	120 feet.
1.	71.50	81.50	91.50	101.50	121.50
1.5	108.37	123.37	138.37	153.37	183.37
2.	146.	166.	186.	206.	246.
2.25	165.09	187.59	210.09	232.59	277.59
2.5	184.37	209.37	234.37	259.37	309.37
2.75	203.84	231.34	258.84	286.34	313.84
3.	223.50	253.50	283.5	313.50	373.50
3.25	243.34	275.84	308.34	340.84	405.84
3.5	263.37	298.37	333 37	368.37	438.37
3.75	283.59	321.09	358.59	396.09	471.09
4.	304.	344.	384.	424.	504.
4.25	324.59	367.09	409.59	452.09	537.09
4.5	345.37	390.37	435.37	480.37	570.37
4.75	366.34	413.84	461.34	508.84	603.84
5.	387.50	437.50	487.50	537.50	637.50
5.25	408.84	461.34	513.84	566.34	671.34
5.5	430.37	485.37	540.37	595.37	705.37
5.75	452.09	509.59	567.09	624.59	739.59
6.	474.	534.	594.	654.	774.
6.25	496.09	558.59	621.09	683.59	808.59
6.5	518.37	583.37	648.37	713.37	843.37
6.75	540.84	608.34	675.84	743.34	878.34
7.	563.50	633.50	703.50	773.50	913.50
7.25	586.34	658.84	731.34	803.84	948.84
7.5	609.37	684.37	759.37	834.37	984.37
7.75	632.59	710.09	787.59	865.09	1020.09
8.	656.	736.	816.	896.	1056.
8.25	679.59	762.09	844.59	927.09	1092.09
8.5	703.37	788.37	873.37	958.37	1128.37
8.75	727.34	814.84	902.34	989.84	1164.84
9.	751.50	841.50	931.50	1021.50	1201.50
9.25	775.84	868.34	960.84	1053.34	1238.34
9.5	800.37	895.37	990.37	985.35	1275.35
9.75	825.09	922.59	1020.09	1117.59	1312.59
10.	850.	950.	1050.	1150.	1350.
10.5	900.37	1005.37	1110.37	1215.37	1425.37
11.	951.50	1061.50	1171.50	1281.50	1501.50
11.5	1003.37	1118.37	1233.37	1348.37	1578.37
12.	1056.	1176.	1296.	1416.	1656.

TABLE 12.

Sectional areas, in square feet, of trapezoidal channels, with side slopes of 1½ to 1

Depth in Feet	Bed Width				
	140 feet.	160 feet.	180 feet.	200 feet.	220 feet.
1.	141.50	161.50	181.50	201.50	221.50
2.	286.	326.	366.	406.	446.
2.5	359.37	409.37	459.37	509.37	559.37
2.75	368.84	423.84	478.84	533.84	588.84
3.	433.50	493.50	553.50	613.50	673.50
3.25	470.80	535.80	600.80	665.80	730.80
3.5	508.37	578.37	648.37	718.47	788.47
3.75	546.09	621.09	696.09	771.09	846.09
4.	584.	664.	744.	824.	904.
4.25	622.09	707.09	792.09	877.09	962.09
4.5	660.37	750.37	840.37	930.37	1020.37
4.75	698.84	793.84	888.84	983.84	1078.84
5.	737.50	837.50	937.50	1037.50	1137.50
5.25	776.34	881.34	986.34	1091.34	1196.34
5.5	815.37	925.37	1035.37	1145.37	1255.37
5.75	854.59	969.59	1084.59	1199.59	1314.59
6.	894.	1014.	1134.	1254.	1374.
6.25	933.59	1058.59	1183.59	1308.59	1433.59
6.5	973.37	1103.37	1233.37	1363.37	1493.37
6.75	1013.34	1148.34	1283.34	1418.34	1553.34
7.	1053.50	1193.50	1333.50	1473.50	1613.50
7.25	1093.84	1238.84	1383.84	1528.84	1673.84
7.5	1134.37	1284.37	1434.37	1584.37	1734.37
7.75	1175.09	1330.09	1485.09	1640.09	1795.09
8.	1216.	1376.	1536.	1696.	1856.
8.25	1257.09	1422.09	1587.09	1752.09	1917.09
8.5	1298.37	1468.37	1638.37	1808.37	1978.37
8.75	1339.84	1514.84	1689.84	1864.84	2039.84
9.	1381.50	1561.50	1741.50	1921.50	2101.50
9.25	1423.34	1608.34	1793.34	1978.34	2163.34
9.5	1465.35	1655.35	1845.35	2035.35	2225.35
9.75	1507.59	1702.59	1897.59	2092.59	2287.59
10.	1550.	1750.	1950.	2150.	2350.
10.5	1635.37	1845.37	2055.37	2265.37	2475.37
11.	1721.50	1941.50	2161.50	2381.50	2601.50
11.5	1808.37	2038.37	2268.37	2498.37	2728.37
12.	1896.	2136.	2376.	2616.	2856.
13.	2073.50	2333.50	2593.50	2853.50	3113.50
14.	2254.	2534.	2814.	3094.	3374.
15.	2437.50	2737.50	3037.50	3337.50	3637.50
16.	2624.	2944.	3264.	3584.	3904.

TABLE 12.

Sectional areas, in square feet, of trapezoidal channels, with side slopes of 1½ to 1.

Depth in Feet.	Bed Width			
	240 feet.	260 feet.	280 feet.	300 feet.
2.	486.	526.	566.	606.
2.5	609.37	659.37	709.37	759.37
3.	733.50	793.50	853.50	913.50
3.25	795.80	860.80	925.80	990.80
3.5	858.47	928.47	998.47	1068.47
3.75	921.09	996.09	1071.09	1146.09
4.	984.	1064.	1144.	1224.
4.25	1047.09	1132.09	1217.09	1302.09
4.5	1110.37	1200.37	1290.37	1380.37
4.75	1173.84	1268.84	1363.84	1458.84
5.	1237.50	1337.50	1437.50	1537.50
5.25	1301.34	1406.34	1511.34	1616.34
5.5	1365.37	1475.37	1585.37	1695.37
5 75	1429.59	1544.59	1659.59	1774.59
6.	1494.	1614.	1734.	1854.
6.25	1558.59	1683.59	1808.59	1933.59
6.5	1623.37	1753.37	1883.37	2013.37
6.75	1688.34	1823.34	1958.34	2093.34
7.	1753.50	1893.50	2033.50	2173.50
7.25	1818.84	1963.84	2108.84	2253.84
7.5	1884.37	2034.37	2184.37	2334.37
7.75	1950.09	2105.09	2260.09	2415.09
8.	2016.	2176.	2336.	2496.
8.25	2181.09	2346.09	2511.09	2676.09
8.5	2148.37	2318.37	2488.37	2658.37
8.75	2214.84	2389.84	2564.84	2739.84
9.	2281.50	2461.50	2641.50	2821.50
9.25	2348.34	2533.34	2718.34	2903.34
9.5	2415.35	2605.35	2795.35	2985.35
9.75	2482.59	2677.59	2872.59	3067.59
10.	2550.	2750.	2950.	3150.
10.5	2685.37	2895.37	3105.37	3315.37
11.0	2821.50	3041.50	3261.50	3481.50
11.5	2958.37	3188.37	3418.37	3648.37
12.	3096.	3336.	3576.	3816.
13.	3373.50	3633.50	3893.50	3153.50
14.	3654.	3934.	4214.	4494.
15.	3937.50	4237.50	4537.50	4837.50
16.	4224.	4544.	4864.	5184.

TABLE 13.

Channels having a rectangular cross-section. Values of the factors a = area in square feet; r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 1 Foot. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 2 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.25	.25	.167	.408	.102	.5	.200	.447	.224	0.25
0.5	.5	.250	.500	.250	1.	.333	.557	.557	0.5
0.75	.75	.300	.548	.411	1.5	.429	.655	.982	0.75
1.	1.	.333	.577	.577	2.	.500	.707	1.414	1.
1.25	1.25	.357	.598	.747	2.5	.555	.744	1.860	1.25
1.5	1.5	.375	.612	.918	3.	.600	.775	2.325	1.5
1.75					3.5	.636	.798	2.793	1.75
2.					4.	.666	.816	3.264	2.
2.25					4.5	.692	.832	3.744	2.25
2.5					5.	.714	.843	4.215	2.5
2.75					5.5	.733	.856	4.708	2.75
3.					6.	.750	.866	5.196	3.
3.25					6.5	.765	.874	5.681	3.25
3.5					7.	.777	.882	6.174	3.5

Depth in Feet.	Bed 3 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 4 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.25	.75	.214	.463	.347	1.	.222	.471	.471	0.25
0.5	1.50	.375	.612	.918	2	.400	.632	1.264	0.5
0.75	2.25	.500	.707	1.591	3.	.545	.738	2.214	0.75
1.	3.	.600	.774	2.322	4.	.666	.816	3.264	1.
1.25	3.75	.682	.825	3.094	5.	.769	.877	4.385	1.25
1.5	4.50	.750	.866	3.897	6.	.857	.926	5.556	1.5
1.75	5.25	.808	.899	4.720	7.	.933	.965	6.755	1.75
2.	6.	.857	.926	5.556	8.	1.	1.	8.	2.
2.25	6.75	.900	.948	6.399	9.	1.058	1.028	9.252	2.25
2.5	7.50	.937	.967	7.252	10.	1.111	1.054	10.540	2.5
2.75	8.25	.971	.989	8.159	11.	1.158	1.076	11.836	2.75
3.	9.	1.	1.	9.	12.	1.200	1.095	13.140	3.
3.5	10.5	1.05	1.024	10.752	14.	1.273	1.128	15.792	3.5
4.	12.	1.091	1.044	12.528	16.	1.333	1.154	18.464	4.
4.5	13.5	1.125	1.067	14.404	18.	1.384	1.185	21.330	4.5
5.	15.	1.154	1.074	16.110	20.	1.428	1.195	23.900	5.

TABLE 13.

Channels having a rectangular cross-section. Values of the factors $a =$ area in square feet; $r =$ hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 5 Feet.				Bed 6 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	2.5	.416	.645	1.612	3.	.428	.654	1.962	0.5
0.75	3.75	.577	.759	2.846	4.5	.600	.775	3.487	0.75
1.	5.	.714	.845	4.225	6.	.750	.866	5.196	1.
1.25	6.25	.833	.913	5.706	7.5	.882	.939	7.042	1.25
1.5	7.5	.937	.968	7.260	9.	1.	1.	9.	1.5
1.75	8.75	1.029	1.014	8.872	10.5	1.106	1.051	11.035	1.75
2.	10.	1.111	1.054	10.540	12.	1.2	1.095	13.140	2.
2.25	11.25	1.184	1.088	12.240	13.5	1.286	1.134	15.309	2.25
2.5	12.5	1.250	1.118	13.975	15.	1.364	1.168	17.520	2.5
2.75	13.75	1.309	1.144	15.730	16.5	1.436	1.198	19.767	2.75
3.	15.	1.364	1.168	17.520	18.	1.5	1.225	22.050	3.
3.25	16.25	1.413	1.187	19.289	19.5	1.56	1.250	24.375	3.25
3.5	17.5	1.458	1.208	21.140	21.	1.615	1.278	26.838	3.5
3.75	18.75	1.500	1.225	22.969	22.5	1.666	1.298	29.205	3.75
4.	20.	1.538	1.241	24.820	24.	1.714	1.309	31.416	4.
4.25	21.25	1.574	1.254	26.647	25.5	1.759	1.326	33.8	4.25
4.5	22.5	1.607	1.268	28.530	27.	1.8	1.341	36.207	4.5
5.	25.	1.666	1.290	32.250	30.	1.875	1.377	41.310	5.

	Bed 7 Feet.				Bed 8 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	3.5	.438	.661	2.313	4.	.444	.667	2.668	0.5
0.75	5.25	.618	.786	4.126	6.	.632	.795	3.792	0.75
1.	7.	.778	.882	6.174	8.	.800	.801	6.408	1.
1.25	8.75	.921	.960	8.400	10.	.857	.826	8.260	1.25
1:5	10.50	1.050	1.025	10.762	12.	1.091	1.044	12.528	1.5
1.75	12.25	1.167	1.080	13.230	14.	1.218	1.104	15.456	1.75
2.	14.	1.273	1.128	15.792	16.	1.333	1.153	18.448	2.
2.25	15.75	1.367	1.170	18.427	18.	1.440	1.200	21.600	2.25
2.5	17.50	1.458	1.208	21.140	20.	1.538	1.240	24.800	2.5
2.75	19.25	1.540	1.241	23.889	22.	1.628	1.276	28.072	2.75
3.	21.	1.615	1.271	26.691	24.	1.714	1.309	31.416	3.
3.25	22.75	1.685	1.298	29.5	26.	1.794	1.340	34.840	3.25
3.5	24.50	1.750	1.323	32.413	28.	1.866	1.366	38.248	3.5
3.75	26.25	1.810	1.345	35.3	30.	1.938	1.392	41.760	3.75
4.	28.	1.866	1.366	38.2	32.	2.	1.414	45.248	4.
4.25	29.75	1.919	1.385	41.2	34.	2.061	1.436	48.824	4.25
4.5	31.50	1.969	1.403	44.1	36.	2.117	1.455	52.380	4.5
4.75	33.25	2.015	1.419	47.2	38.	2.171	1.473	55.974	4.75
5.	35.	2.059	1.435	50.2	40.	2.222	1.490	59.600	5.

TABLE 13.

Channels having a rectangular cross-section. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

Depth in Feet.	Bed 10 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 12 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	10.	.833	.913	9.130	12.	.857	.926	11.112	1.
1.25	12.5	1.	1.	12.50	15.	1.035	1.017	15.255	1.25
1.5	15.	1.154	1.074	16.11	18.	1.2	1.095	19.710	1.5
1.75	17.5	1.295	1.138	19.91	21.	1.357	1.165	24.465	1.75
2.	20.	1.429	1.195	23.90	24.	1.5	1.224	29.376	2.
2.25	22.5	1.553	1.246	28.03	27.	1.636	1.278	34.506	2.25
2.5	25.	1.666	1.290	32.25	30.	1.764	1.328	39.840	2.5
2.75	27.5	1.777	1.333	36.66	33.	1.887	1.374	45.342	2.75
3.	30.	1.875	1.369	41.07	36.	2.	1.414	50.904	3.
3.25	32.5	1.970	1.404	45.63	39.	2.106	1.451	56.589	3.25
3.5	35.	2.058	1.434	50.19	42.	2.209	1.484	62.328	3.5
3.75	37.5	2.143	1.463	54.86	45.	2.304	1.517	68.265	3.75
4.	40.	2.222	1.490	59.60	48.	2.4	1.549	74.352	4.
4.25	42.5	2.297	1.515	64.4	51.	2.488	1.578	80.5	4.25
4.5	45.	2.367	1.538	69.21	54.	2.571	1.603	86.562	4.5
4.75	47.5	2.436	1.561	74.1	57.	2.651	1.628	92.8	4.75
5.	50.	2.5	1.581	79.05	60.	2.727	1.651	99.060	5.
6.	60.	2.727	1.651	99.1	72.	3.000	1.732	124.7	6.

Depth in Feet.	Bed 14 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Bed 16 Feet. a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	14.	.875	.935	13.090	16.	.888	.942	15.072	1.
1.5	21.	1.244	1.115	23.415	24.	1.262	1.123	26.952	1.5
1.75	24.5	1.397	1.182	28.959	28.	1.434	1.197	33.516	1.75
2.	28.	1.555	1.246	34.888	32.	1.600	1.265	40.480	2.
2.25	31.5	1.701	1.304	41.076	36.	1.757	1.325	47.700	2.25
2.5	35.	1.841	1.357	47.495	40.	1.904	1.379	55.160	2.5
2.75	38.5	1.971	1.404	54.054	44.	2.050	1.432	63.008	2.75
3.	42.	2.1	1.450	60.900	48.	2.182	1.455	69.840	3.
3.25	45.5	2.23	1.493	67.931	52.	2.311	1.520	79.040	3.25
3.5	49.	2.333	1.527	74.823	56.	2.346	1.532	85.792	3.5
3.75	52.5	2.447	1.564	82.110	60.	2.556	1.599	95.940	3.75
4.	56.	2.545	1.595	89.320	64.	2.666	1.632	104.448	4.
4.25	59.5	2.644	1.626	96.747	68.	2.774	1.665	113.220	4.25
4.5	63.	2.741	1.655	104.265	72.	2.880	1.697	122.184	4.5
4.75	66.5	2.833	1.683	111.919	76.	2.979	1.726	131.176	4.75
5.	70.	2.917	1.708	119.560	80.	3.080	1.755	140.400	5.
5.5	77.	3.080	1.755	135.135	88.	3.256	1.804	158.752	5.5
6.	84.	3.230	1.797	150.948	96.	3.429	1.852	177.792	6.
6.5	91.	3.367	1.835	166.985	104.	3.588	1.894	196.976	6.5
7.	98.	3.500	1.870	183.260	112.	3.733	1.932	216.384	7.

TABLE 13

Channels having a rectangular cross-section. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 18 Feet.				Bed 20 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
0.5	9.	.526	.725	6.525	10.	.476	.690	6.900	0.5
1.	18.	.900	.948	17.064	20.	.909	.953	19.060	1.
1.5	27.	1.286	1.134	30.620	30.	1.305	1.142	34.260	1.5
2.	36.	1.636	1.279	46.044	40.	1.666	1.290	51.600	2.
2.25	40.5	1.800	1.341	54.310	45.	1.836	1.355	60.975	2.25
2.5	45.	1.953	1.397	62.865	50.	2.	1.414	70.700	2.5
2.75	49.5	2.109	1.452	71.874	55.	2.156	1.468	80.740	2.75
3.	54.	2.250	1.500	81.	60.	2.307	1.518	91.080	3.
3.25	58.5	2.387	1.545	90.382	65.	2.457	1.567	101.855	3.25
3.5	63.	2.520	1.587	99.981	70.	2.590	1.609	112.630	3.5
3.75	67.5	2.646	1.626	109.755	75.	2.727	1.651	123.825	3.75
4.	72.	2.768	1.663	119.736	80.	2.857	1.690	135.200	4.
4.25	76.5	2.892	1.700	130.050	85.	2.975	1.725	146.625	4.25
4.5	81.	3.	1.732	140.292	90.	3.105	1.762	158.580	4.5
4.75	85.5	3.109	1.760	150.480	95.	3.211	1.792	170.240	4.75
5.	90.	3.214	1.792	161.280	100.	3.333	1.825	182.500	5.
5.5	99.	3.416	1.848	182.952	110.	3.553	1.885	207.350	5.5
6.	108.	3.600	1.897	204.876	120.	3.750	1.937	232.440	6.
6.5	117.	3.779	1.944	227.448	130.	3.939	1.984	257.920	6.5
7.	126.	3.938	1.984	249.984	140.	4.116	2.029	284.060	7.

	Bed 25 Feet.				Bed 30 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	25.	.925	.961	24.025	30.	.938	.968	29.040	1.
1.5	37.5	1.338	1.156	43.350	35.	1.364	1.170	40.950	1.5
2.	50.	1.725	1.313	65.650	60.	1.764	1.328	79.680	2.
2.25	56.25	1.901	1.380	77.625	67.5	1.957	1.391	93.892	2.25
2.5	62.5	2.083	1.443	90.187	75.	2.143	1.464	109.800	2.5
2.75	68.75	2.255	1.500	103.125	82.5	2.326	1.525	125.812	2.75
3.	75.	2.422	1.556	116.700	90.	2.500	1.581	142.290	3.
3.25	81.25	2.579	1.606	130.487	97.5	2.672	1.634	159.315	3.25
3.5	87.5	2.734	1.653	144.637	105.	2.835	1.683	176.715	3.5
3.75	93.75	2.884	1.699	159.281	112.5	3.	1.732	194.850	3.75
4.	100.	3.030	1.746	174.600	120.	3.156	1.776	213.120	4.
4.25	106.25	3.166	1.779	189.019	127.5	3.312	1.820	232.050	4.25
4.5	112.5	3.308	1.818	204.525	135.	3.456	1.860	251.100	4.5
4.75	118.75	3.327	1.824	216.600	142.5	3.608	1.899	270.607	4.75
5.	125.	3.571	1.890	236.250	150.	3.750	1.936	290.400	5.
5.5	137 5	3.820	1.954	268.675	165.	4.026	2.006	330.990	5.5
6.	150.	4.050	2.019	302.850	180.	4.286	2.072	372.960	6.
6.5	162.5	4.274	2.057	334.262	195.	4.544	2.131	415.545	6.5
7.	175.	4.480	2.117	370.475	210.	4.773	2.184	458.640	7.
7.5	187.5	4.687	2.165	405.937	225.	5.	2.235	502.875	7.5
8.	200.	4.880	2.209	441.800	240.	5.22	2.284	548.160	8.

TABLE 13.

Channels having a rectangular cross-section. Values of the factors a = area in square feet, and r = hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 35 Feet.				Bed 40 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	35.	.945	.972	34.	40.	.952	.975	39.	1.
1.5	52.5	1.382	1.176	61.7	60.	1.398	1.182	70.9	1.5
2.	70.	1.792	1.338	93.7	80.	1.818	1.348	107.8	2.
2.25	78.75	1.994	1.412	111.2	90.	2.023	1.422	128.	2.25
2.5	87.5	2.187	1.482	129.7	100.	2.222	1.490	149.	2.5
2.75	96.25	2.377	1.542	148.4	110.	2.418	1.555	171.	2.75
3.	105.	2.562	1.600	168.	120.	2.610	1.615	193.8	3.
3.25	113.75	2.741	1.655	188.3	130.	2.795	1.672	217.4	3.25
3.5	122.5	2.919	1.709	209.4	140.	2.982	1.727	241.8	3.5
3.75	131.25	3.071	1.752	229.9	150.	3.099	1.760	264.	3.75
4.	140.	3.162	1.778	248.9	160.	3.333	1.826	292.2	4.
4.25	148.75	3.421	1.849	275.	170.	3.505	1.872	318.2	4.25
4.5	157.5	3.579	1.892	298.	180.	3.672	1.916	344.9	4.5
4.75	166.25	3.737	1.933	321.4	190.	3.838	1.959	372.2	4.75
5.	175.	3.944	1.986	347.6	200.	4.	2.	400.	5.
5.25	183.75	4.038	2.009	369.2	210.	4.158	2.039	428.2	5.25
5.5	192.5	4.177	2.044	389.	220.	4.314	2.077	456.9	5.5
5.75	201.25	4.328	2.080	418.6	230.	4.466	2.113	486.	5.75
6.	210.	4.468	2.114	444.1	240.	4.614	2.148	515.5	6.
6.25	218.75	4.605	2.146	469.4	250.	4.762	2.182	545.5	6.25
6.5	227.5	4.739	2.177	495.3	260.	4.906	2.215	575.9	6.5
6.75	236.25	4.871	2.203	520.5	270.	5.047	2.246	606.4	6.75
7.	245.	5.	2.236	547.8	280.	5.180	2.276	637.3	7.
7.25	253.75	5.126	2.264	574.5	290.	5.321	2.306	668.7	7.25
7.5	262.5	5.250	2.291	601.4	300.	5.455	2.335	700.5	7.5
7.75	271.25	5.372	2.318	628.8	310.	5.586	2.360	731.6	7.75
8.	280.	5.491	2.343	656.	320.	5.714	2.394	766.1	8.
9.	315.	5.943	2.438	768.	360.	6.207	2.491	896.8	9

TABLE 13

Channels having a rectangular section. Values of the factors $a =$ area in square feet, and $r =$ hydraulic mean depth in feet, and also $\sqrt{r}$ and $a\sqrt{r}$ for use in the formulæ

$$v = c \times \sqrt{r} \times \sqrt{s} \text{ and } Q = c \times a\sqrt{r} \times \sqrt{s}$$

	Bed 50 Feet.				Bed 60 Feet.				
Depth in Feet.	a	r	$\sqrt{r}$	$a\sqrt{r}$	a	r	$\sqrt{r}$	$a\sqrt{r}$	Depth in Feet.
1.	50.	.962	.980	49.	60.	.968	.984	59.	1.
2.	100.	1.852	1.360	136.	120.	1.875	1.369	164.3	2.
2.25	112.5	2.063	1.436	161.5	135.	2.093	1.446	195.2	2.25
2.5	125.	2.273	1.507	188.4	150.	2.308	1.519	227.8	2.5
2.75	137.5	2.477	1.574	216.4	165.	2.519	1.587	261.8	2.75
3.	150.	2.679	1.637	245.5	180.	2.727	1.651	297.2	3.
3.25	162.5	2.876	1.696	275.6	195.	2.932	1.712	333.8	3.25
3.5	175.	3.069	1.751	306.4	210.	3.134	1.770	371.7	3.5
3.75	187.5	3.261	1.806	338.6	225.	3.333	1.825	410.6	3.75
4.	200.	3.448	1.857	371.4	240.	3.529	1 878	450.7	4.
4.25	212.5	3.632	1.906	405.	255.	3.722	1.929	491.9	4.25
4.5	225.	3.814	1.953	439.4	270.	3.913	1.978	534.1	4.5
4.75	237.5	3.991	1.997	474.3	285.	4.101	2.025	577.1	4.75
5.	250.	4.167	2.041	510.2	300.	4.286	2.073	621.9	5.
5.25	262.5	4.339	2.083	546.8	315.	4.468	2.114	665.9	5.25
5.5	275.	4.507	2.123	583.8	330.	4.646	2.155	711.1	5.5
5.75	287.5	4.675	2.162	621.6	345.	4.825	2.196	757.6	5.75
6.	300.	4.839	2.200	660.	360.	5.	2.236	805.	6.
6.25	312.5	5.	2.236	698.7	375.	5.172	2.274	852.7	6.25
6.5	325.	5.158	2.271	738.1	390.	5.343	2 311	901.3	6.5
6.75	337.5	5.315	2.305	777.9	405.	5.510	2.347	950.5	6.75
7.	350.	5.470	2.339	818.6	420.	5.676	2.382	1000.4	7.
7.25	362.5	5.620	2.350	851.9	435.	5.839	2.416	1051.	7.25
7.5	375.	5.767	2.401	900.4	450.	6.	2.450	1102.5	7.5
7.75	387.5	5.916	2.432	942.4	465.	6.158	2 481	1153.7	7.75
8.	400.	6.060	2.461	984.4	480.	6.316	2.513	1206.2	8.
8.25	412.5	6.103	2.470	1018.9	495.	6.471	2.544	1259.3	8.25
8.5	425.	6.345	2.519	1070.6	510.	6.624	2.574	1312.7	8.5
8.75	437.5	6.481	2.546	1113.9	525.	6.775	2.603	1366.6	8.75
9.	450.	6.619	2.573	1157.8	540.	6.923	2.633	1421.8	9.
9.25	462.5	6.752	2.598	1201.6	555.	7.010	2.648	1475.7	9.25
9.5	475.	6.883	2.623	1245.9	570.	7.216	2 686	1531.	9.5
9.75	487.5	7.014	2 648	1290.9	585.	7.358	2.712	1586.5	9.75
10.	500.	7.145	2.673	1336.5	600.	7.500	2.738	1642.8	10.
10.5	525.	7.394	2.719	1427	630.	7.778	2.789	1757.	10.5
11.	550.	7.639	2.764	1520	660.	8.049	2.837	1872.4	11.
12.	600.	8.108	2.847	1708.	720.	8.571	2.927	2107 4	12.

TABLE 14. V-SHAPED FLUME, RIGHT-ANGLED CROSS-SECTION.

Based on Kutter's formula, with $n = .013$. Giving values of a, r and c, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The constant factors $c\sqrt{r}$ and $ac\sqrt{r}$ given in table are substantially correct for all slopes up to 1 in 2640, or 2 feet per mile.

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .013$, as in ashlar and well-laid brickwork; ordinary metal; earthenware and stoneware pipe, in good condition but not new; cement and terra cotta pipe, not well jointed nor in perfect order, and also plaster and planed wood in imperfect or inferior condition, and generally the materials mentioned with $n = .01$ when in imperfect or inferior condition

Depth of water in feet.	a = area in square feet.	r = hydraulic mean depth in feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
.40	.16	.141	27.07	4.33
.5	.25	.177	32.54	8.14
.6	.36	.212	37.44	13.48
.7	.49	.247	42.16	20.66
.75	.56	.265	44.55	24.95
.8	.64	.283	46.76	29.92
.9	.81	.318	51.10	41.39
1.	1.	.354	55.63	55.63
1.1	1.21	.389	59.47	72.
1.2	1.44	.424	63.28	91.12
1.25	1.56	.442	65.30	101.9
1.3	1.69	.459	66.40	112 2
1.4	1.96	.494	70.93	139.
1.5	2.25	.530	74.55	167.7
1.6	2.56	.566	78.06	199.8
1.7	2.89	.601	81.53	235.6
1.75	3.06	.618	83.24	254.7
1.8	3.24	.636	85.15	275.9
1.9	3.61	.672	90.47	326.6
2.	4.	.707	91.50	366.
2.1	4.41	.743	94.73	417.8
2.2	4.84	.778	97.90	473.8
2.25	5.06	.795	99.46	503.3
2.3	5.29	.813	101.02	534.4
2.4	5.76	.849	104.	598.9
2.5	6.25	.884	106.9	668.
2.6	6.76	.919	109.9	742.9
2.7	7.29	.955	112.7	821.9
2.75	7.56	.972	114.2	863.2
2.8	7.84	.990	116.2	910.9
2.9	8.41	1.025	118.4	995.8
3.	9.	1.061	121.2	1091.

TABLE 15.

Based on Kutter's formula, with $n = .009$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=0.264 ft. per mile $s = .00005$				1 in 15840=0.3333 ft. per mile $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	93.4	1.49	37.4	1.68	97.8	1.49	39.1	1.72	.4
.5	108.3	1.29	54.2	1.85	112.7	1.27	56.3	1.89	.5
.6	121.2	1.13	72.7	2.	125.4	1.10	75.2	2.03	.6
.7	132.5	.99	92.7	2.12	136.4	.95	95.5	2.13	.7
.8	142.4	.88	113.9	2.22	145.9	.85	116.8	2.21	.8
.9	151.2	.78	136.1	2.29	154.4	.75	138.9	2.30	.9
1.	159.	.71	159.	2.37	161.9	.67	161.9	2.35	1.
1.1	166.1	.64	182.7	2.43	168.6	.60	185.4	2.41	1.1
1.2	172.5	.58	207.	2.48	174.6	.54	209.5	2.45	1.2
1.3	178.3	.53	231.8	2.52	180.	.49	234.	2.49	1.3
1.4	183.6	.48	257.	2.57	184.9	.45	258.9	2.53	1.4
1.5	188.4	.45	282.7	2.59	189.4	.42	284.2	2.55	1.5
1.6	192.9	.41	308.6	2.63	193.6	.38	309.7	2.58	1.6
1.7	197.	.38	334.9	2.66	197.4	.35	335.5	2.60	1.7
1.8	200.8	.35	361.5	2.68	200.9	.32	361.5	2.63	1.8
1.9	204.3	.33	388.3	2.70	204.1	.30	387.8	2.64	1.9
2.	207.6	.31	415.3	2.72	207.1	.28	414.2	2.65	2.
2.1	210.7	.29	442.5	2.74	209.9	.26	440.7	2.67	2.1
2.2	213.6	.27	469.9	2.75	212.5	.24	467.4	2.69	2.2
2.3	216.3	.25	497.4	2.77	214.9	.23	[illegible]	2.70	2.3
2.4	218.8	.24	525.[illegible]	2.78	217.2	.22	521.3	2.70	2.4
2.5	221.2	.22	552.9	2.79	219.3	.20	548.3	2.72	2.5
2.6	223.4	.21	580.8	2.81	221.3	.20	575.[illegible]	2.73	2.6
2.7	225.5	.20	608.9	2.81	223.3	.17	602.8	2.73	2.7
2.8	227.5	.19	637.	2.83	225.	.17	630.[illegible]	2.73	2.8
2.9	229.4	.18	665.3	2.83	226.7	.16	657.[illegible]	2.74	2.9
3	231.2		693.5		228.[illegible]		[illegible]		3

TABLE 15.

Based on Kutter's formula, with $n = .009$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=0.528 ft. per mile				1 in 7500 = 0.704 ft. per mile				$\sqrt{r}$ in feet
	$s = .0001$				$s = .000133333$				
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	105.5	1.47	42.2	1.79	109.5	1.45	43.8	1.82	.4
.5	120.2	1.22	60.1	1.93	124.	1.20	62.	1.96	.5
.6	132.4	1.25	79.4	2.06	136.	1.01	81.6	2.07	.6
.7	142.9	.77	100.	2.15	146.1	.87	102.3	2.15	.7
.8	151.9	.90	121.5	2.22	154.8	.74	123.8	2.22	.8
.9	159.6	.69	143.7	2.28	162.2	.65	146.	2.27	.9
1.	166.5	.60	166.5	2.33	168.7	.57	168.7	2.32	1.
1.1	172.5	.54	189.8	2.41	174.4	.51	191.9	2.35	1.1
1.2	177.9	.48	213.9	2.37	179.5	.45	215.4	2.39	1.2
1.3	182.7	.44	237.6	2.43	184.	.41	239.3	2.40	1.3
1.4	187.1	.39	261.9	2.46	188.1	.37	263.3	2.44	1.4
1.5	191.	.36	286.5	2.49	191.8	.33	287.7	2.45	1.5
1.6	194.6	.33	311.4	2.50	195.1	.31	312.2	2.47	1.6
1.7	197.9	.30	336.4	2.52	198.2	.27	336.9	2.48	1.7
1.8	200.9	.29	361.6	2.54	200.9	.26	361.7	2.49	1.8
1.9	203.8	.24	387.	2.55	203.5	.23	386.6	2.51	1.9
2.	206.2	.24	412.5	2.56	205.8	.22	411.7	2.51	2.
2.1	208.6	.22	438.1	2.57	208.	.20	436.8	2.53	2.1
2.2	210.8	.21	463.8	2.58	210.	.19	462.1	2.53	2.2
2.3	212.9	.19	489.6	2.59	211.9	.18	487.4	2.54	2.3
2.4	214.8	.18	515.5	2.59	213.7	.16	512.8	2.55	2.4
2.5	216.6	.17	541.4	2.61	215.3	.15	538.3	2.55	2.5
2.6	218.3	.15	567.5	2.61	216.8	.15	563.8	2.56	2.6
2.7	219.8	.15	593.6	2.61	218.3	.14	589.4	2.56	2.7
2.8	221.3	.14	619.7	2.63	219.7	.12	615.	2.57	2.8
2.9	222.7	.14	646.	2.66	220.9	.12	640.7	2.57	2.9
3.	224.1		672.6		222.1		666.4		3.

TABLE 15.

Based on Kutter's formula, with $n = .009$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile, $s = .0002$				1 in 3333.3=1.584 ft. per mile, $s = .0003$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	114.1	1.42	45.6	1.86	117.5	1.40	47.	1.88	.4
.5	128.3	1.17	64.2	1.98	131.5	1.14	65.8	2.	.5
.6	140.	.97	84.	2.08	142.9	.95	85.8	2.08	.6
.7	149.7	.83	104.8	2.16	152.4	.79	106.6	2.16	.7
.8	158.	.70	126.4	2.21	160.3	.67	128.2	2.21	.8
.9	165.	.62	148.5	2.27	167.	.59	150.3	2.26	.9
1.	171.2	.53	171.2	2.30	172.9	.51	172.9	2.29	1.
1.1	176.5	.47	194.2	2.33	178.	.44	195.8	2.31	1.1
1.2	181.2	.42	217.5	2.36	182.4	.40	218.9	2.34	1.2
1.3	185.4	.38	241.1	2.38	186.4	.35	242.3	2.36	1.3
1.4	189.2	.34	264.9	2.40	189.9	.32	265.9	2.38	1.4
1.5	192.6	.30	288.9	2.41	193.1	.29	289.7	2.39	1.5
1.6	195.6	.28	313.	2.43	196.	.26	313.6	2.40	1.6
1.7	198.4	.26	337.3	2.44	198.6	.24	337.6	2.42	1.7
1.8	201.	.23	361.7	2.45	201.	.21	361.8	2.42	1.8
1.9	203.3	.21	386.2	2.47	203.1	.20	386.	2.43	1.9
2.	205.4	.20	410.9	2.46	205.1	.19	410.3	2.44	2.
2.1	207.4	.18	435.5	2.48	207.	.17	434.7	2.44	2.1
2.2	209.2	.17	460.3	2.49	208.7	.16	459.1	2.45	2.2
2.3	210.9	.16	485.2	2.49	210.3	.14	483.6	2.45	2.3
2.4	212.5	.15	510.1	2.49	211.7	.14	508.1	2.46	2.4
2.5	214.	.14	535.	2.50	213.1	.13	532.7	2.46	2.5
2.6	215.4	.13	560.	2.50	214.4	.12	557.3	2.47	2.6
2.7	216.7	.12	585.	2.51	215.6	.11	582.	2.47	2.7
2.8	217.9	.11	610.1	2.51	216.7	.11	606.7	2.48	2.8
2.9	219.	.11	635.2	2.52	217.8	.09	631.5	2.47	2.9
3.	220.1		660.4		218.7		656.2		3.

TABLE 15.

Based on Kutter's formula, with $n = .009$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500=2.114 ft. per mile, $s = .0004$				1 in 1000=5.28 ft. per mile, $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	119.3	1.39	47.7	1.89	122.8	1.37	49.1	1.91	.4
.5	133.2	1.13	66.6	1.99	136.5	1.09	68.2	2.02	.5
.6	144.5	.92	86.7	2.09	147.4	.89	88.4	2.10	.6
.7	153.7	.78	107.6	2.16	156.3	.75	109.4	2.16	.7
.8	161.5	.66	129.2	2.21	163.8	.62	131.	2.20	.8
.9	168.1	.57	151.3	2.25	170.	.54	153.	2.24	.9
1.	173.8	.49	173.8	2.28	175.4	.47	175.4	2.27	1.
1.1	178.7	.44	196.6	2.31	180.1	.41	198.1	2.29	1.1
1.2	183.1	.38	219.7	2.33	184.2	.36	221.	2.31	1.2
1.3	186.9	.34	243.	2.35	187.8	.32	244.1	2.33	1.3
1.4	190.3	.31	266.5	2.36	191.	.29	267.4	2.34	1.4
1.5	193.4	.28	290.1	2.38	193.9	.26	290.8	2.36	1.5
1.6	196.2	.25	313.9	2.39	196.5	.23	314.4	2.36	1.6
1.7	198.7	.23	337.8	2.40	198.8	.22	338.	2.38	1.7
1.8	201.	.21	361.8	2.40	201.	.19	361.8	2.37	1.8
1.9	203.1	.19	385.8	2.42	202.9	.18	385.5	2.39	1.9
2.	205.	.18	410.	2.42	204.7	.16	409.4	2.38	2.
2.1	206.8	.16	434.2	2.43	206.3	.15	433.2	2.40	2.1
2.2	208.4	.15	458.5	2.43	207.8	.14	457.2	2.40	2.2
2.3	209.9	.14	482.8	2.44	209.2	.13	481.2	2.40	2.3
2.4	211.3	.13	507.2	2.44	210.5	.13	505.2	2.42	2.4
2.5	212.6	.12	531.6	2.44	211.8	.11	529.4	2.41	2.5
2.6	213.8	.12	556.	2.45	212.9	.11	553.5	2.42	2.6
2.7	215.	.11	580.5	2.45	214.	.10	577.7	2.42	2.7
2.8	216.1	.10	605.	2.46	215.	.09	601.9	2.42	2.8
2.9	217.1	.09	629.6	2.45	215.9	.09	626.1	2.42	2.9
3.	218.		654.1		216.8		650.3		3.

TABLE 16.

Based on Kutter's formula, with $n = .01$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=.264 ft. per mile, $s = .00005$				1 in 15840=.3333 ft. per mile, $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	81.	1.34	32.4	1.48	84.8	1.34	33.9	1.52	.4
.5	94.4	1.16	47.2	1.64	98.2	1.15	49.1	1.67	.5
.6	106.	1.03	63.6	1.78	109.7	1.01	65.8	1.81	.6
.7	116.3	.92	81.4	1.90	119.8	.89	83.9	1.91	.7
.8	125.5	.82	100.4	1.99	128.7	.79	103.	1.99	.8
.9	133.7	.73	120.3	2.07	136.6	.70	122.9	2.07	.9
1.	141.	.66	141.	2.14	143.6	.63	143.6	2.13	1.
1.1	147.6	.61	162.4	2.20	149.9	.57	164.9	2.18	1.1
1.2	153.7	.55	184.4	2.25	155.6	.51	186.7	2.23	1.2
1.3	159.2	.50	206.9	2.30	160.7	.47	209.	2.26	1.3
1.4	164.2	.46	229.9	2.33	165.4	.44	231.6	2.30	1.4
1.5	168.8	.43	253.2	2.38	169.8	.39	254.6	2.33	1.5
1.6	173.1	.40	277.	2.40	173.7	.37	277.9	2.36	1.6
1.7	177.1	.36	301.	2.43	177.4	.33	301.5	2.38	1.7
1.8	180.7	.34	325.3	2.45	180.7	.32	325.3	2.41	1.8
1.9	184.1	.32	349.8	2.48	183.9	.29	349.4	2.42	1.9
2.	187.3	.30	374.6	2.50	186.8	.27	373.6	2.44	2.
2.1	190.3	.28	399.6	2.52	189.5	.25	398.	2.45	2.1
2.2	193.1	.26	424.8	2.53	192.	.24	422.5	2.47	2.2
2.3	195.7	.24	450.1	2.55	194.4	.22	447.2	2.48	2.3
2.4	198.1	.24	475.6	2.56	196.6	.22	472.	2.49	2.4
2.5	200.5	.22	501.2	2.57	198.8	.19	496.9	2.50	2.5
2.6	202.7	.20	526.9	2.59	200.7	.19	521.9	2.51	2.6
2.7	204.7	.20	552.8	2.60	202.6	.18	547.	2.52	2.7
2.8	206.7	.19	578.8	2.61	204.4	.16	572.2	2.53	2.8
2.9	208.6	.17	604.9	2.61	206.	.16	597.5	2.54	2.9
3.	210.3		631.		207.6		622.9		3.

TABLE 16.

Based on Kutter's formula, with $n = .01$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 10000 = .528 ft. per mile				1 in 7500 = .704 ft. per mile				$\sqrt{r}$
	$s = .0001$				$s = .000133333$				
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	91.4	1.34	36.6	1.58	94.9	1.32	38.	1.61	.4
.5	104.8	1.12	52.4	1.72	108.1	1.11	54.1	1.74	.5
.6	116.	.97	69.6	1.84	119.2	.94	71.5	1.85	.6
.7	125.7	.83	88.	1.92	128.6	.80	90.	1.93	.7
.8	134.	.73	107.2	2.	136.6	.70	109.3	2.	.8
.9	141.3	.65	127.2	2.06	143.6	.62	129.3	2.05	.9
1.	147.8	.57	147.8	2.11	149.8	.54	149.8	2.10	1.
1.1	153.5	.52	168.9	2.15	155.2	.49	170.8	2.13	1.1
1.2	158.7	.46	190.4	2.18	160.1	.43	192.1	2.16	1.2
1.3	163.3	.41	212.2	2.22	164.4	.39	213.7	2.19	1.3
1.4	167.4	.38	234.4	2.24	168.3	.36	235.6	2.22	1.4
1.5	171.2	.35	256.8	2.27	171.9	.32	257.8	2.23	1.5
1.6	174.7	.32	279.5	2.29	175.1	.29	280.1	2.26	1.6
1.7	177.9	.29	302.4	2.30	178.	.27	302.7	2.26	1.7
1.8	180.8	.27	325.4	2.32	180.7	.25	325.3	2.28	1.8
1.9	183.5	.25	348.6	2.34	183.2	.23	348.1	2.30	1.9
2.	186.	.23	372.	2.34	185.5	.22	371.1	2.30	2.
2.1	188.3	.22	395.4	2.36	187.7	.20	394.1	2.31	2.1
2.2	190.5	.20	419.	2.37	189.7	.18	417.2	2.32	2.2
2.3	192.5	.19	442.7	2.37	191.5	.17	440.4	2.33	2.3
2.4	194.4	.17	466.4	2.39	193.2	.16	463.7	2.34	2.4
2.5	196.1	.17	490.3	2.39	194.8	.15	487.1	2.34	2.5
2.6	197.8	.15	514.2	2.40	196.3	.15	510.5	2.35	2.6
2.7	199.3	.15	538.2	2.41	197.8	.13	534.	2.35	2.7
2.8	200.8	.14	562.3	2.41	199.1	.13	557.5	3.36	2.8
2.9	202.2	.13	586.4	2.42	200.4	.12	581.1	2.36	2.9
3.	203.5		610.6		201.6		604.7		3.

TABLE 16.

Based on Kutter's formula, with $n = .01$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile $s = .0002$ c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 3333.3=1.584 ft. per mile $s = .0003$ c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	90	1.30	[illegible]	1.64	102.	1.29	40.8	1.67	.4
.5	112	1.06	56	1.77	114.9	1.06	57.5	1.78	.5
.6	122.8	.91	73.7	1.86	125.5	.88	75.3	1.87	.6
.7	131.9	.77	92.3	1.94	134.3	.75	94.	1.94	.7
.8	139.6	.67	111.7	2.	141.8	.64	113.4	1.99	.8
.9	146.3	.58	131.7	2.04	148.2	.55	133.3	2.04	.9
[illegible]	[illegible]	.51	[illegible]	2.06	[illegible]	.51	153.7	2.10	1.
[illegible]	[illegible]	.43	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.08	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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TABLE 16.

Based on Kutter's formula, with $n = .01$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 2500=2.114 ft. per mile. $s = .0004$				1 in 1000 = 5.28 ft. per mile $s = .001$				$\sqrt{r}$
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	103.7	1.28	41.5	1.67	106.9	1.26	42.7	1.70	.4
.5	116.5	1.04	58.2	1.79	119 5	1.01	59.7	1.80	.5
.6	126.9	.87	76.1	1.88	129.6	.84	77.7	1.89	.6
.7	135.6	.73	94.9	1.94	138.	.70	96.6	1.94	.7
.8	142.9	.62	114.3	1.99	145.	.60	116.	1.99	.8
.9	149.1	.55	134.2	2.04	151.	.52	135.9	2.03	.9
1.	154.6	.47	154.6	2.06	156.2	.45	156.2	2.06	1.
1.1	159.3	.42	175.2	2.10	160.7	.39	176.8	2.07	1.1
1.2	163.5	.37	196.2	2.11	164.6	.35	197.5	2.10	1.2
1.3	167.2	.33	217.3	2.14	168.1	.31	218.5	2.11	1.3
1.4	170.5	.28	238.7	2.15	171.2	.28	239.6	2.14	1.4
1.5	173.5	.27	260.2	2.17	174.	.25	261.	2.14	1.5
1.6	176.2	.24	281.9	2.18	176.5	.23	282.4	2.16	1.6
1.7	178.6	.23	303.7	2.19	178.8	.21	304.	2.16	1.7
1.8	180.9	.20	325.6	2.19	180.9	.19	325.6	2.17	1.8
1.9	182.9	.19	347.5	2.21	182.8	.17	347.3	2.17	1.9
2.	184.8	.17	369.6	2.21	184.5	.16	369.	2.18	2.
2.1	186.5	.16	391.7	2.22	186.1	.15	390.8	2.19	2.1
2.2	188.1	.15	413.9	2.23	187.6	.14	412.7	2.20	2.2
2.3	189.6	.14	436.2	2.22	189.	.13	434.7	2.20	2.3
2.4	191.	.13	458.4	2.24	190.3	.12	456.7	2.20	2.4
2.5	192.3	.12	480.8	2.24	191.5	.11	478.7	2.21	2.5
2.6	193.5	.12	503.2	2.24	192.6	.11	500.8	2.22	2.6
2.7	194.7	.10	525.6	2.25	193.7	.10	523.	2.21	2.7
2.8	195.7	.10	548.1	2.24	194.7	.09	545.1	2.21	2.8
2.9	196.7	.10	570.5	2.26	195.6	.08	567.2	2.20	2.9
3.	197.7		593.1		196.4		589.2		3.

TABLE 17.

Based on Kutter's formula, with $n = .011$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000 = .264 ft. per mile, $s = .00005$: c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 15840 = .3333 ft. per mile, $s = .000063131$: c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	71.1	1.22	28.5	1.31	74.5	1.21	29.8	1.35	.4
.5	83.3	1.07	41.6	1.48	86.6	1.06	43.3	1.50	.5
.6	94.	.95	56.4	1.60	97.2	.94	58.3	1.63	.6
.7	103.5	.84	72.4	1.71	106.6	.82	74.6	1.72	.7
.8	111.9	.76	89.5	1.81	114.8	.74	91.8	1.81	.8
.9	119.5	.69	107.6	1.88	122.2	.66	109.9	1.89	.9
1.	126.4	.63	126.4	1.95	128.8	.59	128.8	1.94	1.
1.1	132.7	.57	145.9	2.02	134.7	.54	148.2	1.99	1.1
1.2	138.4	.52	166.1	2.06	140.1	.50	168.1	2.05	1.2
1.3	143.6	.48	186.7	2.11	145.1	.45	188.6	2.08	1.3
1.4	148.4	.44	207.8	2.14	149.6	.41	209.4	2.11	1.4
1.5	152.8	.41	229.2	2.19	153.7	.38	230.5	2.15	1.5
1.6	156.9	.38	251.1	2.21	157.5	.35	252.	2.17	1.6
1.7	160.7	.36	273.2	2.25	161.	.33	273.7	2.20	1.7
1.8	164.3	.33	295.7	2.27	164.3	.30	295.7	2.22	1.8
1.9	167.6	.30	318.4	2.29	167.3	.29	317.9	2.24	1.9
2.	170.6	.29	341.3	2.31	170.2	.26	340.3	2.26	2.
2.1	173.5	.28	364.4	2.34	172.8	.25	362.9	2.27	2.1
2.2	176.3	.25	387.8	2.34	175.3	.23	385.6	2.29	2.2
2.3	178.8	.24	411.2	2.37	177.6	.22	408.5	2.30	2.3
2.4	181.2	.23	434.9	2.39	179.8	.21	431.5	2.31	2.4
2.5	183.5	.21	458.7	2.38	181.9	.19	454.6	2.32	2.5
2.6	185.6	.20	482.6	2.41	183.8	.18	477.8	2.34	2.6
2.7	187.6	.20	506.7	2.41	185.6	.18	501.2	2.34	2.7
2.8	189.6	.18	530.8	2.43	187.4	.16	524.6	2.35	2.8
2.9	191.4	.18	555.1	2.45	189.	.16	548.1	2.36	2.9
3.	193.2		579.6		190.6		571.7		3.

TABLE 17.

Based on Kutter's formula, with $n = .011$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile, $s = .0001$				1 in 7500 = .704 ft. per mile, $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	80.3	1.22	32.1	1.41	83.5	1.21	33.4	1.44	.4
.5	92.5	1.04	46.2	1.55	95.6	1.02	47.8	1.57	.5
.6	102.9	.89	61.7	1.65	105.8	.87	63.5	1.66	.6
.7	111.8	.78	78.2	1.74	114.5	.77	80.1	1.76	.7
.8	119.6	.69	95.6	1.82	122.2	.65	97.7	1.81	.8
.9	126.5	.61	113.8	1.88	128.7	.59	115.8	1.88	.9
1.	132.6	.55	132.6	1.93	134.6	.51	134.6	1.91	1.
1.1	138.1	.49	151.9	1.97	139.7	.47	153.7	1.96	1.1
1.2	143.	.44	171.6	2.	144.4	.41	173.3	1.97	1.2
1.3	147.4	.40	191.6	2.03	148.5	.38	193.	2.02	1.3
1.4	151.4	.37	211.9	2.07	152.3	.34	213.2	2.03	1.4
1.5	155.1	.33	232.6	2.08	155.7	.32	233.5	2.07	1.5
1.6	158.4	.31	253.4	2.11	158.9	.28	254.2	2.07	1.6
1.7	161.5	.28	274.5	2.12	161.7	.27	274.9	2.10	1.7
1.8	164.3	.27	295.7	2.16	164.4	.24	295.9	2.10	1.8
1.9	167.	.24	317.3	2.15	166.8	.22	316.9	2.11	1.9
2.	169.4	.23	338.8	2.18	169.	.21	338.	2.13	2.
2.1	171.7	.21	360.6	2.18	171.1	.20	359.3	2.15	2.1
2.2	173.8	.19	382.4	2.17	173.1	.18	380.8	2.15	2.2
2.3	175.7	.19	404.1	2.21	174.9	.17	402.3	2.15	2.3
2.4	177.6	.17	426.2	2.20	176.6	.16	423.8	2.17	2.4
2.5	179.3	.17	448.2	2.24	178.2	.15	445.5	2.17	2.5
2.6	181.	.15	470.6	2.21	179.7	.14	467.2	2.18	2.6
2.7	182.5	.15	492.7	2.25	181.1	.13	489.	2.17	2.7
2.8	184.	.13	515.2	2.22	182.4	.13	510.7	2.20	2.8
2.9	185.3	.13	537.4	2.24	183.7	.11	532.7	2.17	2.9
3.	186.6		559.8		184.8		554.4		3.

TABLE 17.

Based on Kutter's formula, with $n = .011$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 5000 = 1.056 ft. per mile				1 in 3333.3 = 1.584 ft. per mile				$\sqrt{r}$
	$s = .0002$				$s = .0003$				
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	87.1	1.19	34.8	1.47	89.8	1.18	35.9	1.49	.4
.5	99.	1.	49.5	1.59	101.6	.99	50.8	1.61	.5
.6	109	.85	65.4	1.69	111.5	.82	66.9	1.69	.6
.7	117.5	.73	82.3	1.75	119.7	.71	83.8	1.76	.7
.8	124.8	.63	99.8	1.82	126.8	.61	101.4	1.82	.8
.9	131.1	.55	118.	1.86	132.9	.53	119.6	1.86	.9
1.	136.6	.49	136.6	1.91	138.2	.46	138.2	1.89	1.
1.1	141.5	.44	155.7	1.93	142.8	.41	157.1	1.92	1.1
1.2	145.9	.39	175.	1.97	146.9	.37	176.3	1.95	1.2
1.3	149.8	.35	194.7	1.99	150.6	.34	195.8	1.97	1.3
1.4	153.3	.31	214.6	2.01	154.	.29	215.5	1.99	1.4
1.5	156.4	.29	234.7	2.02	156.9	.28	235.4	2.01	1.5
1.6	159.3	.27	254.9	2.04	159.7	.24	255.5	2.01	1.6
1.7	162	.24	275.3	2.06	162.1	.23	275.6	2.03	1.7
1.8	164.4	.22	295.9	2.07	164.4	.21	295.9	2.04	1.8
1.9	166.6	[illegible]	316.6	[illegible]	166.5	.19	316.3	2.05	1.9
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	168.4	[illegible]	[illegible]	2.06	2
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.1
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.2
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.3
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]		[illegible]		[illegible]		[illegible]		[illegible]

TABLE 17.

Based on Kutter's formula, with $n = .011$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500=2.114 ft. per mile, s = .0004: c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 1000=5.28 ft. per mile, s = .001: c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	91.3	1.18	36.5	1.50	94.1	1.16	37.6	1.52	.4
.5	103.1	.97	51.5	1.62	105.7	.95	52.8	1.63	.5
.6	112.8	.82	67.7	1.70	115.2	.79	69.1	1.71	.6
.7	121.	.69	84.7	1.76	123.1	.67	86.2	1.76	.7
.8	127.9	.59	102.3	1.81	129.8	.57	103.8	1.81	.8
.9	133.8	.52	120.4	1.86	135.5	.49	121.9	1.85	.9
1.	139.	.46	139.	1.89	140.4	.43	140.4	1.88	1.
1.1	143.6	.40	157.9	1.92	144.7	.39	159.2	1.91	1.1
1.2	147.6	.36	177.1	1.94	148.6	.34	178.3	1.93	1.2
1.3	151.2	.32	196.5	1.96	152.	.30	197.6	1.94	1.3
1.4	154.4	.29	216.1	1.98	155.	.27	217.	1.95	1.4
1.5	157.3	.26	235.9	2.	157.7	.25	236.5	1.98	1.5
1.6	159.9	.24	255.9	2.	160.2	.22	256.3	1.98	1.6
1.7	162.3	.22	275.9	2.02	162.4	.20	276.1	1.98	1.7
1.8	164.5	.20	296.1	2.02	164.4	.19	295.9	2.01	1.8
1.9	166.5	.18	316.3	2.04	166.3	.17	316.	2.	1.9
2.	168.3	.18	336.7	2.04	168.	.16	336.	2.01	2.
2.1	170.1	.15	357.1	2.05	169.6	.15	356.1	2.03	2.1
2.2	171.6	.15	377.6	2.05	171.1	.13	376.4	2.01	2.2
2.3	173.1	.14	398.1	2.07	172.4	.13	396.5	2.04	3.3
2.4	174.5	.13	418.8	2.06	173.7	.12	416.9	2.03	2.4
2.5	175.8	.12	439.4	2.07	174.9	.11	437.2	2.04	2.5
2.6	177.	.11	460.1	2.07	176.	.10	457.6	2.04	2.6
2.7	178.1	.10	480.8	2.08	177.	.10	478.	2.04	2.7
2.8	179.1	.10	501.6	2.08	178.	.10	498.4	2.03	2.8
2.9	180.1	.10	522.4	2.08	178.9	.09	518.7	2.06	2.9
3.	181.1		543.2		179.8		539.3		3.

TABLE 18.

Based on Kutter's formula, with $n = .012$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=.264 ft. per mile, $s = .00005$				1 in 15840=.3333 ft. per mile, $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	63.2	1.11	25.3	1.19	66.1	1.12	26.4	1.22	.4
.5	74.3	.98	37.2	1.33	77.3	.98	38.6	1.36	.5
.6	84.1	.88	50.5	1.45	87.1	.86	52.2	1.48	.6
.7	92.9	.79	65.	1.57	95.7	.77	67.	1.57	.7
.8	100.8	.72	80.7	1.65	103.4	.69	82.7	1.66	.8
.9	108.	.64	97.2	1.72	110.3	.63	99.3	1.73	.9
1.	114.4	.59	114.4	1.80	116.6	.56	116.6	1.78	1.
1.1	120.3	.54	132.4	1.85	122.2	.52	134.4	1.85	1.1
1.2	125.7	.50	150.9	1.90	127.4	.47	152.9	1.88	1.2
1.3	130.7	.46	169.9	1.95	132.1	.43	171.7	1.92	1.3
1.4	135.3	.42	189.4	1.99	136.4	.39	190.9	1.95	1.4
1.5	139.5	.40	209.3	2.03	140.3	.37	210.4	2.	1.5
1.6	143.5	.36	229.6	2.05	144.	.34	230.4	2.02	1.6
1.7	147.1	.35	250.1	2.09	147.4	.32	250.6	2.05	1.7
1.8	150.6	.31	271.	2.11	150.6	.29	271.1	2.05	1.8
1.9	153.7	.30	292.1	2.14	153.5	.28	291.6	2.10	1.9
2.	156.7	.29	313.5	2.16	156.3	.27	312.6	2.13	2.
2.1	159.6	.26	335.1	2.17	159.	.23	333.9	2.09	2.1
2.2	162.2	.25	356.8	2.20	161.3	.23	354.8	2.15	2.2
2.3	164.7	.23	378.8	2.21	163.6	.22	376.3	2.16	2.3
2.4	167.	.23	400.9	2.22	165.8	.19	397.9	2.13	2.4
2.5	169.3	.21	423.1	2.25	167.7	.19	419.2	2.18	2.5
2.6	171.4	.20	445.6	2.25	169.6	.18	441.	2.18	2.6
2.7	173.4	.19	468.1	2.26	171.4	.17	462.8	2.19	2.7
2.8	175.3	.18	490.7	2.28	173.1	.17	484.7	2.22	2.8
2.9	177.1	.17	513.5	2.28	174.8	.15	506.9	2.20	2.9
3.	178.8		536.3		176.3		528.9		3.

TABLE 18.

Based on Kutter's formula, with $n = .012$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000 = .528 ft. per mile, $s = .0001$				1 in 7500 = .704 ft. per mile, $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	71.4	1.11	28.5	1.27	74.2	1.11	29.7	1.29	.4
.5	82.5	.96	41.2	1.40	85.3	.95	42.6	1.43	.5
.6	92.1	.84	55.2	1.51	94.8	.82	56.9	1.52	.6
.7	100.5	.74	70.3	1.60	103.	.71	72.1	1.60	.7
.8	107.9	.65	86.3	1.66	110.1	.63	88.1	1.66	.8
.9	114.4	.57	102.9	1.72	116.4	.55	104.7	1.72	.9
1.	120.1	.52	120.1	1.77	121.9	.50	121.9	1.77	1
1.1	125.3	.47	137.8	1.82	126.9	.44	139.6	1.79	1.1
1.2	130.	.42	156.	1.85	131.3	.40	157.5	1.84	1.2
1.3	134.2	.39	174.5	1.88	135.3	.36	175.9	1.86	1.3
1.4	138.1	.35	193.3	1.91	138.9	.33	194.5	1.88	1.4
1.5	141.6	.33	212.4	1.94	142.2	.31	213.3	1.92	1.5
1.6	144.9	.30	231.8	1.96	145.3	.28	232.5	1.93	1.6
1.7	147.9	.27	251.4	1.97	148.1	.25	251.8	1.93	1.7
1.8	150.6	.26	271.1	2.	150.6	.24	271.1	1.96	1.8
1.9	153.2	.24	291.1	2.01	153.	.22	290.7	1.97	1.9
2.	155.6	.22	311.2	2.02	155.2	.21	310.4	1.99	2.
2.1	157.8	.21	331.4	2.04	157.3	.19	330.3	1.99	2.1
2.2	159.9	.19	351.8	2.03	159.2	.18	350.2	2.01	2.2
2.3	161.8	.18	372.1	2.05	161.	.16	370.3	1.99	2.3
2.4	163.6	.17	392.6	2.06	162.6	.16	390.2	2.03	2.4
2.5	165.3	.16	413.2	2.07	164.2	.15	410.5	2.03	2.5
2.6	166.9	.15	433.9	2.08	165.7	.14	430.8	2.04	2.6
2.7	168.4	.15	454.7	2.10	167.1	.13	451.2	2.03	2.7
2.8	169.9	.13	475.7	2.08	168.4	.12	471.5	2.03	2.8
2.9	171.2	.13	496.5	2.10	169.6	.12	491.8	2.06	2.9
3.	172.5		517.5		170.8		512.4		3.

TABLE 18.

Based on Kutter's formula, with $n = .012$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile $s = .0002$				1 in 3333.3=1.584 ft. per mile $s = .0003$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	77.4	1.10	30.9	1.33	79.8	1.10	31.9	1.35	.4
.5	88.4	.94	44.2	1.45	90.8	.92	45.4	1.46	.5
.6	97.8	.79	58.7	1.53	100.	.77	60.	1.54	.6
.7	105.7	.69	74.	1.61	107.7	.67	75.4	1.61	.7
.8	112.6	.60	90.1	1.66	114.4	.58	91.5	1.67	.8
.9	118.6	.53	106.7	1.72	120.2	.51	108.2	1.71	.9
1.	123.9	.46	123.9	1.74	125.3	.44	125.3	1.74	1.
1.1	128.5	.42	141.3	1.79	129.7	.40	142.7	1.77	1.1
1.2	132.7	.38	159.2	1.82	133.7	.36	160.4	1.81	1.2
1.3	136.5	.34	177.4	1.84	137.3	.32	178.5	1.82	1.3
1.4	139.9	.30	195.8	1.85	140.5	.29	196.7	1.84	1.4
1.5	142.9	.28	214.3	1.88	143.4	.27	215.1	1.86	1.5
1.6	145.7	.26	233.1	1.90	146.1	.24	233.7	1.87	1.6
1.7	148.3	.24	252.1	1.92	148.5	.22	252.4	1.88	1.7
1.8	150.7	.21	271.3	1.90	150.7	.20	271.2	1.89	1.8
1.9	152.8	.21	290.3	1.95	152.7	.19	290.1	1.91	1.9
2.	154.9	.18	309.8	1.93	154.6	.18	309.2	1.92	2.
2.1	156.7	.18	329.1	1.96	156.4	.16	328.4	1.92	2.1
2.2	158.5	.16	348.7	1.95	158.	.16	347.6	1.95	2.2
2.3	160.1	.15	368.2	1.96	159.6	.13	367.1	1.90	2.3
2.4	161.6	.14	387.8	1.97	160.9	.13	386.1	1.94	2.4
2.5	163.	.14	407.5	1.99	162.2	.12	405.5	1.93	2.5
2.6	164.4	.12	427.4	1.97	163.4	.12	424.8	1.96	2.6
2.7	165.6	.12	447.1	1.99	164.6	.11	444.4	1.96	2.7
2.8	166.8	.11	467.	1.99	165.7	.10	464.	1.94	2.8
2.9	167.9	.11	486.9	2.01	166.7	.10	483.4	1.97	2.9
3.	169.		507.		167.7		503.1		3.

TABLE 18.

Based on Kutter's formula, with $n = .012$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500=2.114 ft. per mile $s = .0004$				1 in 1000 = 5.28 ft. per mile $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	81.2	1.09	32.5	1.25	83.7	1.09	33.5	1.38	.4
.5	92.1	.91	46.	1.47	94.6	.88	47.3	1.47	.5
.6	101.2	.76	60.7	1.54	103.4	.75	62.	1.56	.6
.7	108.8	.66	76.1	1.62	110.9	.63	77.6	1.61	.7
.8	115.4	.57	92.3	1.67	117.2	.55	93.7	1.67	.8
.9	121.1	.49	109.	1.70	122.7	.47	110.4	1.70	.9
1.	126.	.44	126.	1.74	127.4	.42	127.4	1.74	1.
1.1	130.4	.39	143.4	1.77	131.6	.37	144.8	1.76	1.1
1.2	134.3	.34	161.1	1.79	135.3	.32	162.4	1.76	1.2
1.3	137.7	.31	179.	1.81	138.5	.30	180.	1.81	1.3
1.4	140.8	.29	197.1	1.84	141.5	.26	198.1	1.80	1.4
1.5	143.7	.25	215.5	1.84	144.1	.24	216.1	1.83	1.5
1.6	146.2	.23	233.9	1.85	146.5	.22	234.4	1.84	1.6
1.7	148.5	.22	252.4	1.89	148.7	.20	252.8	1.84	1.7
1.8	150.7	.20	271.3	1.88	150.7	.18	271.2	1.85	1.8
1.9	152.7	.18	290.1	1.89	152.5	.17	289.7	1.87	1.9
2.	154.5	.17	309.	1.90	154.2	.16	308.4	1.88	2.
2.1	156.2	.15	328.	1.89	155.8	.14	327.2	1.86	2.1
2.2	157.7	.15	346.9	1.93	157.2	.14	345.8	1.90	2.2
2.3	159.2	.13	366.2	1.90	158.6	.12	364.8	1.87	2.3
2.4	160.5	.13	385.2	1.93	159.8	.12	383.5	1.90	2.4
2.5	161.8	.12	404.5	1.93	161.	.11	402.5	1.89	2.5
2.6	163.	.11	423.8	1.93	162.1	.10	421.4	1.90	2.6
2.7	164.1	.10	443.1	1.92	163.1	.10	440.4	1.91	2.7
2.8	165.1	.10	462.3	1.94	164.1	.09	459.5	1.90	2.8
2.9	166.1	.09	481.7	1.93	165.	.09	478.5	1.92	2.9
3.	167.		501.		165.9		497.7		3.

TABLE 19.

Based on Kutter's formula, with $n = .013$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=.264 ft. per mile $s = .00005$ c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 15840=.3333 ft. per mile $s = .000063131$ c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	56.7	1.02	22.7	1.08	59.3	1.03	23.7	1.11	.4
.5	66.9	.91	33.5	1.21	69.6	.90	34.8	1.24	.5
.6	76.	.82	45.6	1.33	78.6	.81	47.2	1.35	.6
.7	84.2	.74	58.9	1.44	86.7	.73	60.7	1.45	.7
.8	91.6	.67	73.3	1.51	94.	.65	75.2	1.52	.8
.9	98.3	.61	88.4	1.60	100.5	.59	90.4	1.60	.9
1.	104.4	.56	104.4	1.66	106.4	.53	106.4	1.65	1.
1.1	110.	.52	121.	1.72	111.7	.49	122.9	1.70	1.1
1.2	115.2	.47	138.2	1.76	116.6	.45	139.9	1.76	1.2
1.3	119.9	.44	155.8	1.82	121.1	.41	157.5	1.78	1.3
1.4	124.3	.40	174.	1.85	125.2	.39	175.3	1.84	1.4
1.5	128.3	.38	192.5	1.89	129.1	.35	193.7	1.85	1.5
1.6	132.1	.35	211.4	1.92	132.6	.33	212.2	1.89	1.6
1.7	135.6	.34	230.6	1.96	135.9	.31	231.1	1.91	1.7
1.8	139.	.31	250.2	1.97	139.	.29	250.2	1.93	1.8
1.9	142.1	.28	269.9	2.	141.9	.26	269.5	1.95	1.9
2.	144.9	.29	289.9	2.05	144.5	.25	289.	1.98	2.
2.1	147.8	.25	310.4	2.02	147.	24	308.8	1.98	2.1
2.2	150.3	.24	330.6	2.06	149.4	.22	328.6	2.01	2.2
2.3	152.7	.23	351.2	2.07	151.6	.22	348.7	2.04	2.3
2.4	155.	.21	371.9	2.09	153.8	.19	369.1	2.01	2.4
2.5	157.1	.21	392.8	2.12	155.7	.18	389.2	2.03	2.5
2.6	159.2	.20	414.	2.12	157.5	.18	409.5	2.06	2.6
2.7	161.2	.19	435.2	2.14	159.3	.17	430.1	2.07	2.7
2.8	163.1	.18	456.6	2.16	161.	.16	450.8	2.07	2.8
2.9	164.9	.16	478.2	2.14	162.6	.16	471.5	2.10	2.9
3.	166.5		499.6		164.2		492.5		3.

TABLE 19.

Based on Kutter's formula, with $n = .013$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile $s = .0001$ c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 7500=.704 ft. per mile $s = .000133333$ c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	64.	1.03	25.6	1.16	66.5	1.03	26.6	1.18	.4
.5	74.3	.90	37.2	1.28	76.8	.89	38.4	1.30	.5
.6	83.3	.78	50.	1.38	85.7	.76	51.4	1.39	.6
.7	91.1	.69	63.8	1.46	93.3	.67	65.3	1.47	.7
.8	98.	.61	78.4	1.53	100.	.60	80.	1.54	.8
.9	104.1	.56	93.7	1.60	106.	.53	95.4	1.59	.9
1.	109.7	.49	109.7	1.63	111.3	.47	111.3	1.63	1.
1.1	114.6	.45	126.	1.69	116.	.43	127.6	1.68	1.1
1.2	119.1	.41	142.9	1.72	120.3	.39	144.4	1.70	1.2
1.3	123.2	.37	160.1	1.76	124.2	.35	161.4	1.74	1.3
1.4	126.9	.34	177.7	1.78	127.7	.32	178.8	1.76	1.4
1.5	130.3	.31	195.5	1.80	130.9	.30	196.4	1.78	1.5
1.6	133.4	.29	213.5	1.83	133.9	.27	214.2	1.80	1.6
1.7	136.3	.27	231.8	1.84	136.6	.24	232.2	1.80	1.7
1.8	139.	.25	250.2	1.87	139.	.24	250.2	1.84	1.8
1.9	141.5	.23	268.9	1.88	141.4	.22	268.6	1.85	1.9
2.	143.8	.23	287.7	1.90	143.6	.18	287.1	1.85	2.
2.1	146.1	.19	306.7	1.90	145.4	.20	305.6	1.88	2.1
2.2	148.	.19	325.7	1.92	147.4	.18	324.4	1.87	2.2
2.3	149.9	.18	344.9	1.91	149.2	.15	343.1	1.87	2.3
2.4	151.7	.17	364.	1.95	150.7	.16	361.8	1.90	2.4
2.5	153.4	.15	383.5	1.93	152.3	.15	380.8	1.92	2.5
2.6	154.9	.15	402.8	1.96	153.8	.13	400.	1.89	2.6
2.7	156.4	.15	422.4	1.96	155.1	.14	418.9	1.92	2.7
2.8	157.9	.14	442.	2.	156.5	.12	438.1	1.92	2.8
2.9	159.3	.13	462.	1.98	157.7	.11	457.3	1.91	2.9
3.	160.6		481.8		158.8		476.4		3.

TABLE 19.

Based on Kutter's formula, with $n = .013$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile, $s = .0002$				1 in 3333.3=1.584 ft. per mile, $s = .0003$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	69.4	1.03	27.8	1.20	71.6	1.02	28.6	1.23	.4
.5	79.7	.87	39.8	1.32	81.8	.86	40.9	1.34	.5
.6	88.4	.75	53.	1.41	90.4	.73	54 3	1.41	.6
.7	95.9	.65	67.1	1.48	97.7	.64	68.4	1.49	.7
.8	102.4	.57	81.9	1.54	104.1	.55	83.3	1.45	.8
.9	108.1	.51	97.3	1.59	109.6	.48	97.8	1.66	.9
1.	113.2	.44	113.2	1.61	114.4	.44	114.4	1.63	1.
1.1	117.6	.40	129.3	1.66	118.8	.38	130.7	1.64	1.1
1.2	121.6	.37	145.9	1.69	122.6	.35	147.1	1.68	1.2
1.3	125.3	.32	162.8	1.71	126.1	.31	163.9	1.70	1.3
1.4	128.5	.31	179.9	1.74	129.2	.28	180.9	1.71	1.4
1.5	131.6	.27	197.3	1.75	132.	.26	198.	1.73	1.5
1.6	134.3	.24	214.8	1.76	134.6	.23	215.3	1.74	1.6
1.7	136.7	.24	232.4	1.79	136.9	.22	232.7	1.77	1.7
1.8	139.1	.21	250.3	1.79	139.1	.20	250.4	1.76	1.8
1.9	141.2	.19	268.2	1.81	141.1	.19	268.	1.80	1.9
2.	143.1	.20	286.3	1.83	143.	.16	286.	1.77	2.
2.1	145.1	.16	304.6	1.81	144.6	.17	303.7	1.81	2.1
2 2	146.7	.16	322.7	1.83	146.3	.14	321.8	1.80	2.2
2 3	148.3	.16	341.	1 86	147.7	.14	339 8	1.81	2.3
2 4	149.9	.13	359.6	1.85	149.1	.13	357.9	1.82	2.4
2.5	151.2	.14	378.1	1.85	150.4	.12	376.1	1.83	2.5
2 6	152.6	.12	396.6	1 85	151.7	.13	394.4	1.83	2.6
2.7	153.8	.12	415.1	1.88	152.9	12	412.7	1.82	2.7
2 8	155.	.11	433 9	1.88	153.9	10	430 9	1.85	2.8
2 9	156.1	.10	452.7	1.86	155.	.11	449 4	1.82	2.9
3.	157.1		471 3		155.9	.09	467.6		3.

TABLE 19.

Based on Kutter's formula, with $n = .013$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500 = 2.114 ft. per mile, $s = .0004$				1 in 1000 = 5.28 ft. per mile, $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	72.8	1.02	29.1	1.24	75.2	1.01	30.1	1.25	.4
.5	83.	.85	41.5	1.34	85.3	.83	42.6	1.46	.5
.6	91.5	.73	54.9	1.42	93.6	.71	56.2	1.43	.6
.7	98.8	.62	69.1	1.49	100.7	.60	70.5	1.49	.7
.8	105.	.54	84.	1.54	106.7	.52	85.4	1.53	.8
.9	110.4	.48	99.4	1.58	111.9	.46	100.7	1.57	.9
1.	115.2	.41	115.2	1.61	116.5	.40	116.5	1.60	1.
1.1	119.3	.38	131.3	1.64	120.5	.35	132.5	1.63	1.1
1.2	123.1	.34	147.7	1.67	124.	.32	148.8	1.66	1.2
1.3	126.5	.29	164.4	1.68	127.2	.29	165.4	1.67	1.3
1.4	129.4	.29	181.2	1.72	130.1	.26	182.1	1.69	1.4
1.5	132.3	.24	198.4	1.72	132.7	.23	199.	1.70	1.5
1.6	134.7	.23	215.6	1.73	135.	.21	216.	1.71	1.6
1.7	137.	.21	232.9	1.75	137.1	.20	233.1	1.73	1.7
1.8	139.1	.19	250.4	1.73	139.1	.18	250.4	1.73	1.8
1.9	141.	.18	267.7	1.79	140.9	.17	267.7	1.75	1.9
2.	142.8	.18	285.6	1.80	142.6	.15	285.2	1.74	2.
2.1	144.6	.14	303.6	1.77	144.1	.14	302.6	1.75	2.1
2.2	146.	.14	321.3	1.77	145.5	.14	320.1	1.78	2.2
2.3	147.4	.14	339.	1.81	146.9	.12	337.9	1.75	2.3
2.4	148.8	.12	357.1	1.79	148.1	.11	355.4	1.77	2.4
2.5	150.	.12	375.	1.81	149.2	.11	373.1	1.78	2.5
2.6	151.2	.11	393.1	1.81	150.3	.10	390.9	1.77	2.6
2.7	152.3	.10	411.2	1.80	151.3	.10	408.6	1.79	2.7
2.8	153.3	.10	429.2	1.83	152.3	.09	426.5	1.79	2.8
2.9	154.3	.09	447.5	1.81	153.2	.09	444.4	1.80	2.9
3.	155.2		465.6		154.1		462.4		3.

TABLE 20.

Based on Kutter's formula, with $n = .015$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000 = .264 ft. per mile, $s = .00005$				1 in 15840 = .3333 ft. per mile, $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	46.8	.87	18.7	.91	48.9	.88	19.6	.93	.4
.5	55.5	.79	27.8	1.02	57.7	.79	28.9	1.05	.5
.6	63.4	.70	38.	1.13	65.6	.69	39.4	1.14	.6
.7	70.4	.67	49.3	1.24	72.5	.66	50.8	1.25	.7
.8	77.1	.60	61.7	1.31	79.1	.58	63.3	1.31	.8
.9	83.1	.55	74.8	1.38	84.9	.53	76.4	1.38	.9
1.	88.6	.50	88.6	1.44	90.2	.49	90.2	1.45	1.
1.1	93.6	.47	103.	1.50	95.1	.45	104.7	1.49	1.1
1.2	98.3	.44	118.	1.55	99.6	.42	119.6	1.53	1.2
1.3	102.7	.40	133.5	1.59	103.8	.38	134.9	1.57	1.3
1.4	106.7	.38	149.4	1.63	107.6	.36	150.6	1.61	1.4
1.5	110.5	.35	165.7	1.67	111.2	.33	166.7	1.64	1.5
1.6	114.	.33	182.4	1.70	114.5	.31	183.1	1.68	1.6
1.7	117.3	.31	199.4	1.73	117.6	.28	199.9	1.68	1.7
1.8	120.4	.29	216.7	1.76	120.4	.27	216.7	1.72	1.8
1.9	123.3	.28	234.3	1.78	123.1	.26	233.9	1.74	1.9
2.	126.1	.25	252.1	1.80	125.7	.24	251.3	1.77	2.
2.1	128.6	.25	270.1	1.83	128.1	.22	269.	1.77	2.1
2.2	131.1	.23	288.4	1.84	130.3	.21	286.7	1.79	2.2
2.3	133.4	.22	306.8	1.87	132.4	.21	304.6	1.82	2.3
2.4	135.6	.21	325.5	1.87	134.5	.18	322.8	1.81	2.4
2.5	137.7	.20	344.2	1.91	136.3	.19	340.9	1.83	2.5
2.6	139.7	.19	363.3	1.91	138.2	.17	359.2	1.87	2.6
2.7	141.6	.18	382.4	1.91	139.9	.17	377.9	1.85	2.7
2.8	143.4	.17	401.5	1.93	141.6	.15	396.4	1.87	2.8
2.9	145.1	.16	420.8	1.94	143.1	.14	415.1	1.85	2.9
3.	146.7		440.2		144.5		433.6		3.

TABLE 20.

Based on Kutter's formula, with $n = .015$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile $s = .0001$				1 in 7500 = .704 ft. per mile $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	52.7	.89	21.1	.97	54.7	.90	21.9	.99	.4
.5	61.6	.78	30.8	1.09	63.7	.78	31.8	1.11	.5
.6	69.4	.68	41.7	1.17	71.5	.67	42.9	1.18	.6
.7	76.2	.63	53.4	1.26	78.2	.61	54.7	1.28	.7
.8	82.5	.56	66.	1.33	84.3	.54	67.5	1.32	.8
.9	88.1	.50	79.3	1.38	89.7	.48	80.7	1.38	.9
1.	93.1	.46	93.1	1.43	94.5	.44	94.5	1.43	1.
1.1	97.7	.41	107.4	1.47	98.9	.39	108.8	1.46	1.1
1.2	101.8	.38	122.1	1.51	102.8	.37	123.4	1.50	1.2
1.3	105.6	.34	137.2	1.54	106.5	.32	138.4	1.52	1.3
1.4	109.	.32	152.6	1.57	109.7	.30	153.6	1.55	1.4
1.5	112.2	.30	168.3	1.60	112.7	.28	169.1	1.58	1.5
1.6	115.2	.27	184.3	1.62	115.5	.26	184.9	1.59	1.6
1.7	117.9	.25	200.5	1.63	118.1	.24	200.8	1.61	1.7
1.8	120.4	.25	216.8	1.66	120.5	.22	216.9	1.62	1.8
1.9	122.9	.21	233.4	1.67	122.7	.21	233.1	1.65	1.9
2.	125.	.21	250.1	1.69	124.8	.19	249.6	1.65	2.
2.1	127.1	.20	267.	1.70	126.7	.18	266.1	1.66	2.1
2.2	129.1	.18	284.	1.70	128.5	.17	282.7	1.68	2.2
2.3	130.9	.17	301.	1.73	130.2	.16	299.5	1.68	2.3
2.4	132.6	.17	318.3	1.74	131.8	.15	316.3	1.69	2.4
2.5	134.3	.15	335.7	1.75	133.3	.14	333.2	1.70	2.5
2.6	135.8	.15	353.2	1.75	134.7	.13	350.2	1.71	2.6
2.7	137.3	.14	370.7	1.76	136.	.13	367.3	1.72	2.7
2.8	138.7	.13	388.3	1.78	137.3	.12	384.5	1.72	2.8
2.9	140.	.12	406.1	1.75	138.5	.12	401.7	1.69	2.9
3.	141.2		423.6		139.7		558.6		3.

TABLE 20.

Based on Kutter's formula, with $n = .015$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 5000=1.056 ft. per mile				1 in 3333.3=1.584 ft. per mile				$\sqrt{r}$
	$s = .0002$				$s = .0003$				
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	57.1	.90	22.9	1.01	59.	.89	23.6	1.03	.4
.5	66.1	.77	33.	1.13	67.9	.76	33.9	1.14	.5
.6	73.8	.65	44.3	1.19	75.5	.65	45.3	1.21	.6
.7	80.3	.60	56.2	1.29	82.	.58	57.4	1.28	.7
.8	86.3	.52	69.1	1.33	87.8	.50	70.2	1.33	.8
.9	91.5	.46	82.4	1.37	92.8	.45	83.5	1.38	.9
1.	96.1	.42	96.1	1.42	97.3	.40	97.3	1.41	1.
1.1	100.3	.37	110.3	1.45	101.3	.36	111.4	1.45	1.1
1.2	104.	.34	124.8	1.48	104.9	.32	125.9	1.47	1.2
1.3	107.4	.31	139.6	1.51	108.1	.30	140.6	1.49	1.3
1.4	110.5	.28	154.7	1.53	111.1	.26	155.5	1.51	1.4
1.5	113.3	.26	170.	1.54	113.7	.25	170.6	1.53	1.5
1.6	115.9	.24	185.4	1.57	116.2	.22	185.9	1.54	1.6
1.7	118.3	.22	201.1	1.58	118.4	.21	201.3	1.56	1.7
1.8	120.5	.21	216.9	1.60	120.5	.20	216.9	1.58	1.8
1.9	122.6	.19	232.9	1.60	122.5	.17	232.7	1.58	1.9
2.	[illegible]	[illegible]	248.9	[illegible]	[illegible]	[illegible]	248.5	1.59	2.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	264.3	1.59	2.1
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.2
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.3
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	2.6
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.7
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.8
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.9
3	[illegible]		[illegible]		[illegible]		[illegible]		3

TABLE 20.

Based on Kutter's formula, with $n = .015$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500=2.114 ft. per mile, $s = .0004$				1 in 1000=5.28 ft. per mile, $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	60.	.89	24.	1.04	62.	.88	24.8	1.06	.4
.5	68.9	.75	34.4	1.15	70.8	.75	35.4	1.16	.5
.6	76.4	.64	45.9	1.21	78.3	.63	47.	1.22	.6
.7	82.8	.58	58.	1.29	84.6	.55	59.2	1.29	.7
.8	88.6	.50	70.9	1.33	90.1	.48	72.1	1.33	.8
.9	93.6	.43	84.2	1.37	94 9	.42	85.4	1.37	.9
1.	97.9	.39	97.9	1.41	99.1	.38	99.1	1.41	1.
1.1	101.8	.35	112.	1.44	102.9	.33	113.2	1.42	1.1
1.2	105.3	.32	126.4	1.46	106.2	.30	127.4	1.46	1.2
1.3	108.5	.28	141.	1.49	109.2	.27	142.	1.47	1.3
1.4	111.3	.27	155.9	1.50	111.9	.25	156.7	1.49	1.4
1.5	114.	.23	170.9	1.52	114.4	.22	171.6	1.50	1.5
1.6	116.3	.22	186.1	1.54	116.6	.20	186.6	1.50	1.6
1.7	118.5	.20	201.5	1.55	118.6	.19	201.6	1.53	1.7
1.8	120.5	.19	217.	1.55	120.5	.18	216.9	1.55	1.8
1.9	122.4	.17	232.5	1.56	122.3	.16	232.4	1.54	1.9
2.	124.1	.16	248.1	1.59	123.9	.15	247.8	1.55	2.
2.1	125.7	.15	264.	1.59	125.4	.14	263.3	1.57	2.1
2.2	127.2	.14	279.9	1.59	126.8	.12	279.	1.54	2.2
2.3	128.6	.13	295.8	1.59	128.	.13	294.4	1.59	2.3
2.4	129.9	.12	311.7	1.60	129.3	.11	310.3	1.57	2.4
2.5	131.1	.12	327.7	1.43	130.4	.10	326.	1.56	2.5
2.6	132.3	.10	342.	1.80	131.4	.11	341.6	1.62	2.6
2.7	133.3	.11	360.	1.63	132.5	.09	357.8	1.57	2.7
2.8	134.4	.10	376.3	1.63	133.4	.09	373.5	1.60	2.8
2.9	135.4	.08	392.6	1.61	134.3	.08	389.5	1.58	2.9
3.	136.2		408.7		135.1		405.3		3.

TABLE 21.

Based on Kutter's formula, with $n = .017$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

	1 in 20000=.264 ft. per mile				1 in 15840=.3333 ft. per mile				
$\sqrt{r}$	$s = .00005$				$s = .000063131$				$\sqrt{r}$
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	39.6	.76	15.9	.77	41.3	.77	16.5	.80	.4
.5	47.2	.70	23.6	.89	49.	.70	24.5	.91	.5
.6	54.2	.63	32.5	.99	56.	.63	33.6	1.	.6
.7	60.5	.59	42.4	1.07	62.3	.58	43.6	1.09	.7
.8	66.4	.54	53.1	1.15	68.1	.52	54.5	1.15	.8
.9	71.8	.49	64.6	1.21	73.3	.49	66.	1.22	.9
1.	76.7	.47	76.7	1.28	78.2	.45	78.2	1.27	1.
1.1	81.4	.43	89.5	1.33	82.7	.41	90.9	1.32	1.1
1.2	85.7	.40	102.8	1.38	86.8	.38	104.1	1.37	1.2
1.3	89.7	.37	116.6	1.42	90.6	.36	117.8	1.40	1.3
1.4	93.4	.35	130.8	1.46	94.2	.33	131.8	1.44	1.4
1.5	96.9	.33	145.4	1.49	97.5	.31	146.2	1.47	1.5
1.6	100.2	.31	160.3	1.53	100.6	.29	160.9	1.51	1.6
1.7	103.3	.29	175.6	1.56	103.5	.27	176.	1.52	1.7
1.8	106.2	.28	191.2	1.59	106.2	.26	191.2	1.55	1.8
1.9	109.	.26	[illegible]	1.61	108.8	.24	[illegible]	1.58	1.9
2.	[illegible]	.24	[illegible]	1.63	[illegible]	.23	[illegible]	1.59	2.
2.1	[illegible]	.24	[illegible]	[illegible]	[illegible]	.22	[illegible]	1.61	2.1
2.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.62	2.2
2.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.3
2.4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.6
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.7
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.8
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.9
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 21.

Based on Kutter's formula, with $n = .017$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile $s = .0001$				1 in 7500=704 ft. per mile $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	44.5	.78	17.8	.83	46.2	.78	18.5	.85	.4
.5	52.3	.69	26.1	.94	54.	.70	27.0	.96	.5
.6	59.2	.62	35.5	1.03	61.	.61	36.6	1.04	.6
.7	65.4	.57	45.8	1.11	67.1	.55	47.0	1.11	.7
.8	71.1	.50	56.9	1.16	72.6	.49	58.1	1.16	.8
.9	76.1	.46	68.5	1.22	77.5	.44	69.7	1.22	.9
1.	80.7	.41	80.7	1.26	81.9	.41	81.9	1.27	1.
1.1	84.8	.39	93.3	1.31	86.	.36	94.6	1.29	1.1
1.2	88.7	.35	106.4	1.35	89.6	.34	107.5	1.34	1.2
1.3	92.2	.32	119.9	1.37	93.	.31	120.9	1.36	1.3
1.4	95.4	.31	133.6	1.42	96.1	.28	134.5	1.38	1.4
1.5	98.5	.27	147.8	1.41	98.9	.27	148.3	1.42	1.5
1.6	101.2	.26	161.9	1.46	101.6	.24	162.5	1.43	1.6
1.7	103.8	.25	176.5	1.48	104.	.23	176.8	1.45	1.7
1.8	106.3	.22	191.3	1.48	106.3	.21	191.3	1.47	1.8
1.9	108.5	.21	206.1	1.51	108.4	.20	206.	1.48	1.9
2.	110.6	.21	221.2	1.54	110.4	.18	220.8	1.48	2.
2.1	112.7	.18	236.6	1.53	112.2	.18	235.6	1.52	2.1
2.2	114.5	.18	251.9	1.56	114.	.16	250.8	1.51	2.2
2.3	116.3	.17	267.5	1.57	115.6	.16	265.9	1.54	2.3
2.4	118.	.16	283.2	1.58	117.2	.14	281.3	1.52	2.4
2.5	119.6	.15	299.	1.58	118.6	.14	296.5	1.55	2.5
2.6	121.1	.14	314.8	1.59	120.	.13	312.	1.55	2.6
2.7	122.5	.13	330.7	1.59	121.3	.13	327.5	1.58	2.7
2.8	123.8	.13	346.6	1.62	122.6	.11	343.3	1.54	2.8
2.9	125.1	.12	362.8	1.61	123.7	.12	358.7	1.60	2.9
3.	126.3	.12	378.9	1.66	124.9	.10	374.7	1.56	3.
3.1	127.5	.11	395.3	1.62	125.9	.10	390.3	1.58	3.1
3.2	128.6	.11	411.5	1.65	126.9	.10	406.1	1.60	3.2
3.3	129.7	.10	428.	1.64	127.9	.09	422.1	1.58	3.3
3.4	130.7	.10	444.4	1.65	128.8	.09	437.9	1.61	3.4
3.5	131.7	.09	460.9	1.65	129.7	.09	454.	1.62	3.5
3.6	132.6	.09	477.4	1.65	130.6	.08	470.2	1.60	3.6
3.7	133.5	.08	493.9	1.64	131.4	.07	486.2	1.58	3.7
3.8	134.3	.09	510.3	1.70	132.1	.08	502.	1.63	3.8
3.9	135.2	.08	527.3	1.65	132.9	.07	518.3	1.61	3.9
4.	136.		543.8		133.6		534.4		4.

TABLE 21.

Based on Kutter's formula, with $n = .017$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile $s = .0002$				1 in 3333.3=1.584 ft. per mile $s = .0003$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	48.2	.79	19.3	.87	49.8	.78	19.9	.89	.4
.5	56.1	.68	28.	.98	57.6	.68	28.8	.98	.5
.6	62.9	.61	37.8	1.05	64.4	.60	38.6	1.07	.6
.7	69.	.53	48.3	1.11	70.4	.52	49.3	1.12	.7
.8	74.3	.48	59.4	1.18	75.6	.46	60.5	1.17	.8
.9	79.1	.42	71.2	1.21	80.2	.42	72.2	1.22	.9
1.	83.3	.39	83.3	1.22	84.4	.37	84.4	1.25	1.
1.1	87.2	.35	95.9	1.29	88.1	.34	96.9	1.29	1.1
1.2	90.7	.32	108.8	1.32	91.5	.30	109.8	1.30	1.2
1.3	93.9	.29	122.	1.35	94.5	.28	122.8	1.34	1.3
1.4	96.8	.26	135.5	1.37	97.3	.25	136.2	1.35	1.4
1.5	99.4	.25	149.2	1.38	99.8	.24	149.7	1.38	1.5
1.6	101.9	.23	163.	1.41	102.2	.21	163.5	1.38	1.6
1.7	104.2	.21	177.1	1.43	104.3	.20	177.3	1.40	1.7
1.8	106.3	.20	191.4	1.43	106.3	.19	191.3	1.43	1.8
1.9	108.3	.18	205.7	1.45	108.2	.17	205.6	1.42	1.9
2.	110.1	.17	220.2	1.46	109.9	.16	219.8	1.43	2.
2.1	111.7	.16	234.8	1.47	111.5	.15	234.1	1.45	2.1
2.2	113.4	.15	249.5	1.48	113.	.14	248.6	1.45	2.2
2.3	114.9	.14	264.3	1.49	114.4	.13	263.1	1.46	2.3
2.4	116.3	.14	279.2	1.50	115.7	.13	277.7	1.48	2.4
2.5	117.7	.12	294.2	1.50	117.	.11	292.5	1.46	2.5
2.6	118.9	.12	309.2	1.51	118.1	.12	307.1	1.50	2.6
2.7	120.1	.11	324.3	1.51	119.3	.10	322.1	1.47	2.7
2.8	121.2	.11	339.4	1.52	120.3	.10	336.8	1.50	2.8
2.9	122.3	.10	354.6	1.53	121.3	.09	351.8	1.48	2.9
3.	123.3	.10	369.9	1.53	122.2	.09	366.6	1.50	3.
3.1	124.3	.09	385.2	1.54	123.1	.08	381.6	1.49	3.1
3.2	125.2	.09	400.6	1.54	123.9	.08	396.5	1·50	3.2
3.3	126.1	.08	416.	1.54	124.7	.08	411.5	1.52	3.3
3.4	126.9	.08	431.4	1.54	125.5	.07	426.7	1.50	3.4
3.5	127.7	.08	446.8	1.58	126.2	.07	441.7	1.51	3.5
3.6	128.5	.07	462.6	1.53	126.9	.07	456.8	1.53	3.6
3.7	129.2	.06	477.9	1.55	127.6	.06	472.1	1.51	3.7
3.8	129.8	.07	493.4	1.57	128.2	.07	487.2	1.55	3.8
3.9	130.5	.07	509.1	1.55	128.9	.06	502.7	1.55	3.9
4.	131.2		524.6		129.5		518.2		4.

TABLE 21.

Based on Kutter's formula, with $n = .017$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500 = 2.114 ft. per mile, $s = .0004$				1 in 1666.7 = 3.168 ft. per mile, $s = .0006$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	50.5	.78	20.2	.90	51.5	.79	20.6	.91	.4
.5	58.3	.68	29.2	.99	59.4	.67	29.7	1.00	.5
.6	65.1	.59	39.1	1.06	66.1	.58	39.7	1.07	.6
.7	71.	.52	49.7	1.12	71.9	.51	50.4	1.12	.7
.8	76.2	.45	60.9	1.18	77.	.45	61.6	1.18	.8
.9	80.7	.41	72.7	1.21	81.5	.40	73.4	1.21	.9
1.	84.8	.37	84.8	1.25	85.5	.36	85.5	1.25	1.
1.1	88.5	.32	97.3	1.28	89.1	.31	98.	1.27	1.1
1.2	91.7	.30	110.1	1.30	92.2	.30	110.7	1.31	1.2
1.3	94.7	.27	123.1	1.33	95.2	.26	123.8	1.32	1.3
1.4	97.4	.25	136.4	1.35	97.8	.24	137.	1.33	1.4
1.5	99.9	.23	149.9	1.36	100.2	.22	150.3	1.36	1.5
1.6	102.2	.21	163.5	1.38	102.4	.21	163.9	1.37	1.6
1.7	104.3	.19	177.3	1.39	104.5	.21	177.6	1.38	1.7
1.8	106.2	.18	191.2	1.40	106.3	.18	191.4	1.39	1.8
1.9	108.	.17	205.2	1.42	108.1	.16	205.3	1.40	1.9
2.	109.7	.15	219.4	1.42	109.7	.15	219.3	1.42	2.
2.1	111.2	.15	233.6	1.43	111.2	.14	233.5	1.41	2.1
2.2	112.7	.14	247.9	1.44	112.6	.13	247.6	1.44	2.2
2.3	114.1	.12	262.3	1.45	113.9	.12	262.	1.42	2.3
2.4	115.3	.12	276.8	1.45	115.1	.11	276.2	1.44	2.4
2.5	116.5	.11	291.3	1.46	116.2	.11	290.6	1.44	2.5
2.6	117.6	.11	305.9	1.46	117.3	.10	305.	1.45	2.6
2.7	118.7	.10	320.5	1.47	118.3	.10	319.5	1.45	2.7
2.8	119.7	.10	335.2	1.47	119.3	.09	334.	1.46	2.8
2.9	120.7	.09	349.9	1.48	120.2	.09	348.6	1.46	2.9
3.	121.6	.08	364.7	1.48	121.1	.08	363.2	1.46	3.
3.1	122.4	.08	379.5	1.49	121.9	.08	377.8	1.47	3.1
3.2	123.2	.08	394.4	1.48	122.7	.07	392.5	1.47	3.2
3.3	124.	.08	409.2	1.51	123.4	.07	407.2	1.47	3.3
3.4	124.8	.07	424.3	1.48	124.1	.07	421.9	1.48	3.4
3.5	125.5	.06	439.1	1.50	124.8	.06	436.7	1.48	3.5
3.6	126.1	.07	454.1	1.51	125.4	.06	451.5	1.47	3.6
3.7	126.8	.06	469.2	1.52	126.	.06	466.2	1.48	3.7
3.8	127.4	.06	484.4	1.47	126.6	.06	481.	1.49	3.8
3.9	128.	.05	499.1	1.50	127.2	.05	495.9	1.49	3.9
4.	128.5		514.1		127.7		510.8		4.

TABLE 21.

Based on Kutter's formula, with $n = .017$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250 = 4.224 ft. per mile, $s = .0008$				1 in 1000 = 5.28 ft. per mile, $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	52.	.78	20.8	.91	52.3	.78	20.9	.91	.4
.5	59.8	.68	29.9	1.	60.1	.67	30.	1.01	.5
.6	66.6	.58	39.9	1.08	66.8	.58	40.1	1.07	.6
.7	72.4	.50	50.7	1.12	72.6	.51	50.8	1.14	.7
.8	77.4	.45	61.9	1.18	77.7	.44	62.2	1.17	.8
.9	81.9	.39	73.7	1.21	82.1	.39	73.9	1.21	.9
1.	85.8	.34	85.8	1.23	86.	.35	86.	1.24	1.
1.1	89.2	.33	98.1	1.29	89.5	.31	98.4	1.27	1.1
1.2	92.5	.29	111.	1.30	92.6	.29	111.1	1.30	1.2
1.3	95.4	.26	124.	1.32	95.5	.26	124.1	1.32	1.3
1.4	98.	.23	137.2	1.33	98.1	.23	137.3	1.33	1.4
1.5	100.3	.22	150.5	1.35	100.4	.21	150.6	1.34	1.5
1.6	102.5	.20	164.	1.37	102.5	.20	164.	1.36	1.6
1.7	104.5	.18	177.7	1.37	104.5	.18	177.6	1.37	1.7
1.8	106.3	.17	191.4	1.39	106.3	.17	191.3	1.39	1.8
1.9	108.	.16	205.3	1.39	108.	.16	205.2	1.40	1.9
2.	109.6	.15	219.2	1.41	109.6	.14	219.2	1.39	2.
2.1	111.1	.13	233.3	1.41	111.	.14	233.1	1.42	2.1
2.2	112.4	.13	247.4	1.42	112.4	.12	247.3	1.40	2.2
2.3	113.7	.12	261.6	1.42	113.6	.12	261.3	1.42	2.3
2.4	114.9	.11	275.8	1.43	114.8	.12	275.5	1.48	2.4
2.5	116.	.11	290.1	1.44	116.1	.13	290.3	1.40	2.5
2.6	117.1	.10	304.5	1.44	117.1	.10	304.3	1.46	2.6
2.7	118.1	.09	318.9	1.44	118.1	.10	318.9	1.43	2.7
2.8	119.	.09	333.3	1.45	119.	.09	333.2	1.45	2.8
2.9	119.9	.09	347.8	1.45	119.9	.09	347.7	1.44	2.9
3.	120.8	.08	362.3	1.46	120.7	.08	362.1	1.46	3.
3.1	121.6	.07	376.9	1.44	121.5	.08	376.7	1.43	3.1
3.2	122.3	.07	391.3	1.47	122.2	.07	391.	1.46	3.2
3.3	123.	.07	406.	1.47	122.9	.07	405.6	1.46	3.3
3.4	123.7	.07	420.7	1.47	123.6	.07	420.2	1.49	3.4
3.5	124.4	.06	435.4	1.46	124.3	.07	435.1	1.45	3.5
3.6	125.	.07	450.	1.51	124.9	.06	449.6	1.47	3.6
3.7	125.7	.05	465.1	1.44	125.5	.06	464.3	1.48	3.7
3.8	126.2	.05	479.5	1.47	126.1	.06	479.1	1.46	3.8
3.9	126.7	.05	494.2	1.48	126.6	.05	493.7	1.47	3.9
4.	127.2		509.		127.1	.05	508.4		4.

TABLE 22.

Based on Kutter's formula, with $n = .02$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=.264 ft. per mile $s = .00005$				1 in 15840 = .3333 ft. per mile $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	32.	.63	12.8	.64	33.3	.65	13.3	.66	.4
.5	38.3	.59	19.2	.73	39.8	.58	19.9	.75	.5
.6	44.2	.54	26.5	.82	45.6	.55	27.4	.83	.6
.7	49.6	.51	34.7	.91	51.1	.49	35.7	.91	.7
.8	54.7	.47	43.8	.96	56.	.46	44.8	.98	.8
.9	59.4	.43	53.4	1.03	60.6	.43	54.6	1.03	.9
1.	63.7	.41	63.7	1.09	64.9	.39	64.9	1.08	1.
1.1	67.8	.38	74.6	1.13	68.8	.38	75.7	1.14	1.1
1.2	71.6	.36	85.9	1.18	72.6	.34	87.1	1.17	1.2
1.3	75.2	.34	97.7	1.23	76.	.32	98.8	1.21	1.3
1.4	78.6	.31	110.	1.26	79.2	.30	110.9	1.25	1.4
1.5	81.7	.31	122.6	1.30	82.2	.29	123.4	1.28	1.5
1.6	84.8	.28	135.6	1.33	85.1	.27	136.2	1.30	1.6
1.7	87.6	.26	148.9	1.35	87.8	.25	149.2	1.33	1.7
1.8	90.2	.26	162.4	1.40	90.3	.24	162.5	1.36	1.8
1.9	92.8	.24	176.4	1.40	92.7	.22	176.1	1.38	1.9
2.	95.2	.23	190.4	1.44	94.9	.22	189.9	1.39	2.
2.1	97.5	.22	204.8	1.44	97.1	.20	203.8	1.42	2.1
2.2	99.7	.21	219.4	1.46	99.1	.19	218.	1.43	2.2
2.3	101.8	.20	234.2	1.49	101.	.18	232.3	1.45	2.3
2.4	103.8	.19	249.1	1.51	102.8	.18	246.8	1.47	2.4
2.5	105.7	.18	264.2	1.53	104.6	.17	261.5	1.49	2.5
2.6	107.5	.18	279.5	1.55	106.3	.16	276.4	1.48	2.6
2.7	109.3	.16	295.	1.54	107.9	.15	291.2	1.51	2.7
2.8	110.9	.15	310.4	1.57	109.4	.14	306.3	1.51	2.8
2.9	112.4	.16	326.1	1.59	110.8	.14	321.4	1.53	2.9
3.	114.	.15	342.	1.61	112.2	.14	336.7	1.55	3.
3.1	115.5	.14	358.1	1.60	113.6	.13	352.2	1.55	3.1
3.2	116.9	.14	374.1	1.59	114.9	.12	367.7	1.54	3.2
3.3	118.3	.13	390.	1.66	116.1	.12	383.1	1.57	3.3
3.4	119.6	.12	406.6	1.62	117.3	.11	398.8	1.56	3.4
3.5	120.8	.12	422.8	1.64	118.4	.11	414.4	1.58	3.5
3.6	122.	.12	439.2	1.66	119.5	.11	430.2	1.60	3.6
3.7	123.2	.11	455.8	1.65	120.6	.10	446.2	1.59	3.7
3.8	124.3	.11	472.3	1.68	121.6	.09	462.1	1.56	3.8
3.9	125.4	.11	489.1	1.69	122.5	.10	477.7	1.63	3.9
4.	126.5		506.		123.5		494.		4.

TABLE 22.

Based on Kutter's formula, with $n = .02$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile. $s = .0001$				1 in 7500 = .704 ft. per mile $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	35.7	.66	14.3	.69	37.1	.66	14.8	.70	.4
.5	42.3	.59	21.2	.77	43.7	.59	21.8	.80	.5
.6	48.2	.54	28.9	.86	49.6	.53	29.8	.86	.6
.7	53.6	.48	37.5	.92	54.9	.48	38.4	.94	.7
.8	58.4	.45	46.7	.99	59.7	.43	47.8	.98	.8
.9	62.9	.40	56.6	1.03	64.	.40	57.6	1.04	.9
1.	66.9	.38	66.9	1.08	68.	.36	68.	1.08	1.
1.1	70.7	34	77.7	1.13	71.6	.33	78.8	1.11	1.1
1.2	74.1	.32	89.	1.15	74.9	.31	89.9	1.15	1.2
1.3	77.3	.30	100.5	1.19	78.	.28	101.4	1.18	1.3
1.4	80.3	.28	112.4	1.22	80.8	.27	113.2	1.20	1.4
1.5	83.1	.25	124.6	1.24	83.5	.24	125.2	1.23	1.5
1.6	85.6	.24	137.	1.27	85.9	.23	137.5	1.24	1.6
1.7	88.	.23	149.7	1.29	88.2	.21	149.9	1.27	1.7
1.8	90.3	.21	162.6	1.30	90.3	.20	162.6	1.28	1.8
1.9	92.4	.20	175.6	1.33	92.3	.19	175.4	1.30	1.9
2.	94.4	.19	188.9	1.33	94.2	.17	188.4	1.31	2.
2.1	96.3	.18	202.2	1.36	95.9	.17	201.5	1.32	2.1
2.2	98.1	.17	215.8	1.37	97.6	.16	214.7	1.34	2.2
2.3	99.8	.16	229.5	1.38	99.2	.15	228.1	1.25	2.3
2.4	101.4	.15	243.3	1.39	100.7	.14	241.6	1.35	2.4
2.5	102.9	.14	257.2	1.40	102.1	.13	255.1	1.37	2.5
2.6	104.3	.14	271.2	1.41	103.4	.12	268.8	1.37	2.6
2.7	105.7	.13	285.3	1.43	104.6	.12	282.5	1.39	2.7
2.8	107.	.12	299.6	1.43	105.8	.12	296.4	1.38	2.8
2.9	108.2	.12	313.9	1.43	107.	.11	310.2	1.40	2.9
3.	109.4	.11	328.2	1.45	108.1	.10	324 2	1.40	3.
3.1	110.5	.11	342.7	1.45	109.1	.10	338.2	1.41	3.1
3.2	111.6	.11	357.2	1.46	110.1	.09	352.3	1.41	3.2
3.3	112.7	.10	371.8	1.46	111.	.09	366.4	1.42	3.3
3.4	113.7	.09	386.4	1.47	111.9	.09	380.6	1.43	3.4
3.5	114.6	.09	401.1	1.47	112.8	.09	394.9	1.42	3.5
3.6	115.5	.09	415.8	1.49	113.7	.08	409.1	1.46	3.6
3.7	116.4	.08	430.7	1.47	114.5	.07	423.7	1.41	3.7
3.8	117.2	.08	445.4	1.48	115.2	.07	437.8	1.44	3.8
3.9	118.	.08	460.2	1.51	115.9	.08	452.2	1.45	3.9
4.	118.8		475.3		116.7		466.7		4.

TABLE 22.

Based on Kutter's formula, with $n = .02$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile, $s = .0002$: c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 3333.3=1.584 ft. per mile, $s = .0003$: c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	38.7	.66	15.5	.72	39.9	.67	16.	.73	.4
.5	45.3	.59	22.7	.80	46.6	.58	23.3	.82	.5
.6	51.2	.52	30.7	.88	52.4	.52	31.5	.88	.6
.7	56.4	.47	39.5	.94	57.6	.46	40.3	.95	.7
.8	61.1	.43	48.9	.99	62.2	.42	49.8	.99	.8
.9	65.4	.38	58.8	1.04	66.4	.37	59.7	1.04	.9
1.	69.2	.35	69.2	1.07	70.1	.33	70.1	1.07	1.
1.1	72.7	.31	79.9	1.11	73.4	.31	80.8	1.10	1.1
1.2	75.8	.30	91.	1.14	76.5	.28	91.8	1.13	1.2
1.3	78.8	.26	102.4	1.16	79.3	.26	103.1	1.16	1.3
1.4	81.4	.25	114.	1.19	81.9	.23	114.7	1.17	1.4
1.5	83.9	.23	125.9	1.21	84.2	.22	126.4	1.19	1.5
1.6	86.2	.22	138.	1.22	86.4	.20	138.3	1.21	1.6
1.7	88.4	.19	150.2	1.24	88.4	.20	150.4	1.23	1.7
1.8	90.3	.19	162.6	1.26	90.4	.17	162.7	1.24	1.8
1.9	92.2	.17	175.2	1.27	92.1	.16	175.1	1.24	1.9
2.	93.9	.17	187.9	1.28	93.7	.16	187.5	1.26	2.
2.1	95.6	.15	200.7	1.29	95.3	.14	200.1	1.27	2.1
2.2	97.1	.14	213.6	1.30	96.7	.14	212.8	1.28	2.2
2.3	98.5	.14	226.6	1.31	98.1	.12	225.6	1.27	2.3
2.4	99.9	.13	239.7	1.32	99.3	.12	238.3	1.30	2.4
2.5	101.2	.12	252.9	1.33	100.5	.12	251.3	1.30	2.5
2.6	102.4	.11	266.2	1.33	101.7	.10	264.3	1.30	2.6
2.7	103.5	.11	279.5	1.34	102.7	.10	277.3	1.32	2.7
2.8	104.6	.10	292.9	1.35	103.7	.10	290.5	1.31	2.8
2.9	105.6	.10	306.4	1.35	104.7	.09	303.6	1.32	2.9
3.	106.6	.10	319.9	1.36	105.6	.09	316.8	1.33	3.
3.1	107.6	.09	333.5	1.36	106.5	.08	330.1	1.33	3.1
3.2	108.5	.08	347.1	1.37	107.3	.08	343.4	1.34	3.2
3.3	109.3	.08	360.8	1.37	108.1	.68	356.8	1.34	3.3
3.4	110.1	.08	374.5	1.37	108.9	.07	370.2	1.34	3.4
3.5	110.9	.08	388.2	1.38	109.6	.07	383.6	1.34	3.5
3.6	111.7	.07	402.	1.39	110.3	.07	397.	1.35	3.6
3.7	112.4	.07	415.9	1.39	111.	.06	410.5	1.35	3.7
3.8	113.1	.06	429.8	1.43	111.6	.06	424.	1.35	3.8
3.9	113.7	.07	443.6	1.39	112.2	.06	437.5	1.36	3.9
4.	114.4		457.5		112.8		451.1		4.

TABLE 22.

Based on Kutter's formula, with $n = .02$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500 = 2.112 ft. per mile, $s = .0004$				1 in 1666.7 = 3.168 ft. per mile, $s = .0006$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	40.6	.67	16.2	.74	41.3	.67	16.5	.75	.4
.5	47.3	.58	23.6	.83	48.	.58	24.	.83	.5
.6	53.1	.51	31.9	.89	53.8	.51	32.3	.90	.6
.7	58.2	.46	40.8	.95	58.9	.45	41.3	.94	.7
.8	62.8	.41	50.3	.99	63.4	.41	50.7	1	.8
.9	66.9	.37	60.2	1.04	67.5	.35	60.7	1.03	.9
1.	70.6	.32	70.6	1.06	71.	.33	71.	1.07	1.
1.1	73.8	.31	81.2	1.10	74.3	.30	81.7	1.10	1.1
1.2	76.9	.27	92.2	1.13	77.3	.26	92.7	1.12	1.2
1.3	79.6	.25	103.5	1.15	79.9	.25	103.9	1.14	1.3
1.4	82.1	.23	115.	1.17	82.4	.22	115.3	1.16	1.4
1.5	84.4	.22	126.7	1.18	84.6	.21	126.9	1.21	1.5
1.6	86.6	.19	138.5	1.20	86.7	.19	138.7	1.19	1.6
1.7	88.5	.18	150.5	1.21	88.6	.18	150.6	1.21	1.7
1.8	90.3	.17	162.6	1.23	90.4	.16	162.7	1.21	1.8
1.9	92	.16	174.9	1.24	92	.15	174.8	1.23	1.9
2.	[illegible]	.15	[illegible]	1.25	93.5	.15	[illegible]	[illegible]	2
2.1	[illegible]	.15	199.8	1.25	95	.13	[illegible]	[illegible]	2.1
2.2	96.6	[illegible]	[illegible]	[illegible]	96.3	[illegible]	[illegible]	[illegible]	2.2
2.3	97.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.3
2.4	99	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.2
3.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	106.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	109.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]		[illegible]				[illegible]		[illegible]

TABLE 22.

Based on Kutter's formula, with $n = .02$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250=4.224 ft. per mile $s = .0008$				1 in 1000 = 5.28 ft. per mile $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	41.7	.67	16.7	.75	41.9	.67	16.8	.75	.4
.5	48.4	.58	24.2	.83	48.6	.58	24.3	.83	.5
.6	54.2	.51	32.5	.90	54.4	.51	32.6	.90	.6
.7	59.3	.45	41.5	.95	59.5	.45	41.6	.96	.7
.8	63.8	.40	51.	1.	64.	.40	51.2	1.00	.8
.9	67.8	.35	61.	1.03	68.	.35	61.2	1.03	.9
1.	71.3	.32	71.3	1.07	71.5	.32	71.5	1.08	1.
1.1	74.5	.29	82.	1.09	74.7	.29	82.3	1.08	1.1
1.2	77.4	.27	92.9	1.12	77.6	.26	93.1	1.12	1.2
1.3	80.1	.24	104.1	1.14	80.2	.24	104.3	1.13	1.3
1.4	82.5	.22	115.5	1.16	82.6	.22	115.6	1.16	1.4
1.5	84.7	.20	127.1	1.17	84.8	.20	127.2	1.17	1.5
1.6	86.7	.19	138.8	1.18	86.8	.18	138.9	1.17	1.6
1.7	88.6	.18	150.6	1.21	88.6	.18	150.6	1.21	1.7
1.8	90.4	.16	162.7	1.21	90.4	.16	162.7	1.21	1.8
1.9	92.	.15	174.8	1.22	92.	.15	174.8	1.22	1.9
2.	93.5	.14	187.	1.22	93.5	.13	187.	1.21	2.
2.1	94.9	.13	199.2	1.25	94.8	.13	199.1	1.23	2.1
2.2	96.2	.13	211.7	1.25	96.1	.13	211.4	1.26	2.2
2.3	97.5	.11	224.2	1.24	97.4	.11	224.	1.24	2.3
2.4	98.6	.11	236.6	1.26	98.5	.11	236.4	1.26	2.4
2.5	99.7	.10	249.2	1.27	99.6	.10	249.	1.25	2.5
2.6	100.7	.10	261.9	1.27	100.6	.09	261.5	1.26	2.6
2.7	101.7	.09	274.6	1.27	101.5	.09	274.1	1.26	2.7
2.8	102.6	.09	287.3	1.28	102.4	.09	286.7	1.29	2.8
2.9	103.5	.08	300.1	1.27	103.3	.08	299.6	1.27	2.9
3.	104.3	.08	312.8	1.30	104.1	.08	312.3	1.29	3.
3.1	105.1	.07	325.8	1.28	104.9	.07	325.2	1.27	3.1
3.2	105.8	.07	338.6	1.28	105.6	.07	337.9	1.29	3.2
3.3	106.5	.07	351.4	1.31	106.3	.07	350.8	1.30	3.3
3.4	107.2	.07	364.5	1.31	107.	.06	363.8	1.28	3.4
3.5	107.9	.06	377.6	1.30	107.6	.06	376.6	1.29	3.5
3.6	108.5	.06	390.6	1.31	108.2	.06	389.5	1.31	3.6
3.7	109.1	.05	403.7	1.28	108.8	.06	402.6	1.31	3.7
3.8	109.6	.06	416.5	1.33	109.4	.05	415.7	1.29	3.8
3.9	110.2	.05	429.8	1.30	109.9	.05	428.6	1.30	3.9
4.	110.7		442.8		110.4		441.6		4.

TABLE 23.

Based on Kutter's formula, with $n = .0225$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=.264 ft. per mile $s = .00005$ c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 15840=.3333 ft. per mile $s = .000063131$ c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	27.4	.78	11.	.50	28.5	.57	11.4	.57	.4
.5	33.	.52	16.5	.64	34.2	.52	17.1	.65	.5
.6	38.2	.48	22.9	.72	39.4	.48	23.6	.74	.6
.7	43.	.45	30.1	.79	44.2	.45	31.	.79	.7
.8	47.5	.42	38.	.85	48.7	.41	38.9	.87	.8
.9	51.7	.40	46.5	.92	52.8	.39	47.6	.91	.9
1.	55.7	.37	55.7	.96	56.7	.36	56.7	.96	1.
1.1	59.4	.36	65.3	1.03	60.3	.34	66.3	1.02	1.1
1.2	63.	.32	75.6	1.05	63.7	.32	76.5	1.05	1.2
1.3	66.2	.31	86.1	1.09	66.9	.30	87.	1.08	1.3
1.4	69.3	.30	97.	1.15	69.9	.28	97.8	1.13	1.4
1.5	72.3	.28	108.5	1.17	72.7	.27	109.1	1.15	1.5
1.6	75.1	.26	120.2	1.19	75.4	.25	120.6	1.18	1.6
1.7	77.7	.25	132.1	1.23	77.9	.23	132.4	1.20	1.7
1.8	80.2	.24	144.4	1.25	80.2	.23	144.4	1.23	1.8
1.9	82.6	.23	156.9	1.29	82.5	.21	156.7	1.25	1.9
2.	84.9	.22	169.8	1.31	84.6	.21	169.2	1.25	2.
2.1	87.1	.20	182.9	1.31	86.7	.19	182.	1.28	2.1
2.2	89.1	.20	196.	1.35	88.6	.18	194.9	1.31	2.2
2.3	91.1	.19	209.5	1.37	90.4	.18	208.	1.32	2.3
2.4	93.	.18	223.2	1.38	92.2	.17	221.2	1.35	2.4
2.5	94.8	.18	237.	1.42	93.9	.16	234.7	1.35	2.5
2.6	96.6	.16	251.2	1.39	95.5	.15	248.2	1.37	2.6
2.7	98.2	.16	265.1	1.43	97.	.15	261.9	1.38	2.7
2.8	99.8	.16	279.4	1.47	98.5	.14	275.7	1.39	2.8
2.9	101.4	.14	294.1	1.43	99.9	.13	289.6	1.41	2.9
3.	102.8	.15	308.4	1.49	101.2	.13	303.7	1.41	3.
3.1	104.3	.13	323.3	1.46	102.5	.13	317.8	1.42	3.1
3.2	105.6	.13	337.9	1.52	103.8	.12	332.	1.44	3.2
3.3	106.9	.13	353.1	1.49	105.	.11	346.4	1.44	3.3
3.4	108.2	.13	368.	1.51	106.1	.11	360.8	1.45	3.4
3.5	109.5	.12	383.1	1.53	107.2	.11	375.3	1.46	3.5
3.6	110.7	.11	398.4	1.53	108.3	.10	389.9	1.47	3.6
3.7	111.8	.11	413.7	1.53	109.3	.10	404.6	1.47	3.7
3.8	112.9	.11	429.	1.55	110.3	.10	419.3	1.48	3.8
3.9	114.	.10	444.5	1.56	111.3	.09	434.1	1.48	3.9
4.	115.		460.1		112.2		448.9		4.

TABLE 23.

Based on Kutter's formula, with $n = .0225$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile $s = .0001$				1 in 7500 = .704 ft. per mile $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	30.5	.58	12.2	.60	31.6	.59	12.6	.61	.4
.5	36.3	.53	18.2	.68	37.5	.52	18.7	.69	.5
.6	41.6	.48	25.	.75	42.7	.48	25.6	.77	.6
.7	46.4	.43	32.5	.81	47.5	.43	33.3	.81	.7
.8	50.7	.41	40.6	.87	51.8	.40	41.4	.88	.8
.9	54.8	.37	49.3	.92	55.8	.36	50.2	.92	.9
1.	58.5	.34	58.5	.96	59.4	.33	59.4	.96	1.
1.1	61.9	.32	68.1	1.	62.7	.31	69.	1.	1.1
1.2	65.1	.30	78.1	1.04	65.8	.29	79.	1.03	1.2
1.3	68.1	.27	88.5	1.06	68.7	.26	89.3	1.05	1.3
1.4	70.8	.26	99.1	1.10	71.3	.25	99.8	1.09	1.4
1.5	73.4	.25	110.1	1.13	73.8	.23	110.7	1.11	1.5
1.6	75.9	.22	121.4	1.14	76.1	.22	121.8	1.13	1.6
1.7	78.1	.22	132.8	1.17	78.3	.20	133.1	1.14	1.7
1.8	80.3	.20	144.5	1.19	80.3	.18	144.5	1.17	1.8
1.9	82.3	.19	156.4	1.20	82.1	.16	156.2	1.18	1.9
2.	84.2	.18	168.4	1.22	83.7	.16	168.	1.18	2.
2.1	86.	.17	180.6	1.23	85.3	.15	179.8	1.20	2.1
2.2	87.7	.16	192.9	1.25	86.8	.13	191.8	1.24	2.2
2.3	89.3	.15	205.4	1.25	88.1	.14	204.2	1.23	3.3
2.4	90.8	.15	217.9	1.29	89.5	.12	216.5	1.23	2.4
2.5	92.3	.14	230.8	1.28	90.7	.12	228.8	1.25	2.5
2.6	93.7	.13	243.6	1.29	91.9	.11	241.3	1.25	2.6
2.7	95.	.13	256.5	1.31	93.	.11	253.8	1.28	2.7
2.8	96.3	.12	269.6	1.32	94.1	.10	266.6	1.27	2.8
2.9	97.5	.11	282.8	1.30	95.1	.10	279.3	1.29	2.9
3.	98.6	.11	295.8	1.33	96.1	.09	292.2	1.28	3.
3.1	99.7	.11	309.1	1.35	97.	.09	305.	1.31	3.1
3.2	100.8	.10	322.6	1.33	97.9	.08	318.1	1.29	3.2
3.3	101.8	.10	335.9	1.36	98.7	.08	331.	1.31	3.3
3.4	102.8	.09	349.5	1.35	99.5	.08	344.1	1.29	3.4
3.5	103.7	.09	363.	1.36	100.3	.07	357.	1.31	3.5
3.6	104.6	.09	376.6	1.38	101.	.07	370.1	1.32	3.6
3.7	105.5	.08	390.4	1.35	101.7	.07	383.3	1.34	3.7
3.8	106.3	.08	403.9	1.38	102.4	.07	396.7	1.36	3.8
3.9	107.1	.08	417.7	1.39	103.1	.06	410.3	1.37	3.9
4.	107.9		431.6		103.7		424.		4.

TABLE 23

Based on Kutter's formula, with $n = .0225$. Values of the factors c and $c\sqrt{r}$ for use in the formulae

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

	1 in 5000 1.056 ft. per mile				1 in 3333.3=1.584 ft. per mile				
$\sqrt{r}$	$s = .0002$				$s = .0003$				$\sqrt{r}$
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	33.	.58	13.2	.62	34.	.59	13.6	.64	.4
.5	38.8	.53	19.4	.71	39.9	.53	20.	.71	.5
.6	44.1	.48	26.5	.77	45.2	.46	27.1	.78	.6
.7	48.9	.42	34.2	.83	49.8	.44	34.9	.85	.7
.8	53.1	.38	42.5	.87	54.2	.36	43.4	.86	.8
.9	56.9	.36	51.2	.93	57.8	.34	52.	.92	.9
1.	60.5	.32	60.5	.96	61.2	.32	61.2	.96	1.
1.1	63.7	.29	70.1	.98	64.4	.28	70.8	.98	1.1
1.2	66.6	.27	79.9	1.02	67.2	.26	80.6	1.01	1.2
1.3	69.3	.26	90.1	1.06	69.8	.25	90.7	1.05	1.3
1.4	71.9	.23	100.7	1.06	72.3	.22	101.2	1.06	1.4
1.5	74.2	.22	111.3	1.09	74.5	.21	111.8	1.08	1.5
1.6	76.4	.20	122.2	1.11	76.6	.19	122.6	1.09	1.6
1.7	78.4	.19	133.3	1.12	78.5	.17	133.5	1.09	1.7
1.8	80.3	.18	144.5	1.15	80.2	[illegible]	144.4	1.12	1.8
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 23.

Based on Kutter's formula, with $n = .0225$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500=2.112 ft. per mile, $s = .0004$: c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 1666.6=3.168 ft. per mile, $s = .0006$: c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	34.6	.59	13.8	.65	35.2	.59	14.1	.65	.4
.5	40.5	.52	20.3	.71	41.1	.52	20.6	.75	.5
.6	45.7	.47	27.4	.79	46.3	.47	28.1	.76	.6
.7	50.4	.41	35.3	.83	51.	.41	35.7	.84	.7
.8	54.5	.38	43.6	.89	55.1	.37	44.1	.88	.8
.9	58.3	.34	52.5	.92	58.8	.33	52.9	.92	.9
1.	61.7	.30	61.7	.95	62.1	.30	62.1	.95	1.
1.1	64.7	.28	71.2	.98	65.1	.28	71.6	.99	1.1
1.2	67.5	.26	81.	1.01	67.9	.25	81.5	1.	1.2
1.3	70.1	.24	91.1	1.04	70.4	.23	91.5	1.03	1.3
1.4	72.5	.22	101.5	1.04	72.7	.21	101.8	1.07	1.4
1.5	74.7	.20	111.9	1.08	74.8	.20	112.5	1.05	1.5
1.6	76.7	.19	122.7	1.09	76.8	.18	123.	1.08	1.6
1.7	78.6	.17	133.6	1.09	78.6	.17	133.8	1.07	1.7
1.8	80.3	.16	144.5	1.11	80.3	.16	144.5	1.11	1.8
1.9	81.9	.16	155.6	1.14	81.9	.15	155.6	1.12	1.9
2.	83.5	.14	167.	1.13	83.4	.13	166.8	1.09	2.
2.1	84.9	.13	178.3	1.13	84.7	.13	177.7	1.11	2.1
2.2	86.2	.13	189.6	1.17	86.	.13	188.8	1.15	2.2
2.3	87.5	.12	201.3	1.16	87.3	.11	200.3	1.14	2.3
2.4	88.7	.11	212.9	1.16	88.4	.11	211.7	1.13	2.4
2.5	89.8	.11	224.5	1.18	89.5	.10	223.	1.15	2.5
2.6	90.9	.10	236.3	1.18	90.5	.10	234.5	1.17	2.6
2.7	91.9	.09	248.1	1.17	91.5	.09	246.2	1.14	2.7
2.8	92.8	.09	259.8	1.19	92.4	.09	257.6	1.18	2.8
2.9	93.7	.09	271.7	1.21	93.3	.08	269.4	1.29	2.9
3.	94.6	.08	283.8	1.19	94.1	.08	282.3	1.19	3.
3.1	95.4	.08	295.7	1.21	94.9	.07	294.2	1.04	3.1
3.2	96.2	.08	307.8	1.23	95.6	.08	304.6	1.19	3.2
3.3	97.	.07	320.1	1.21	96.4	.06	316.5	1.16	3.3
3.4	97.7	.07	332.2	1.22	97.	.07	328.1	1.20	3.4
3.5	98.4	.06	344.4	1.20	97.7	.06	340.1	1.18	3.5
3.6	99.	.06	356.4	1.23	98.3	.06	351.9	1.20	3.6
3.7	99.6	.06	368.7	1.23	98.9	.06	363.9	1.19	3.7
3.8	100.2	.06	381.	1.22	99.5	.06	375.8	1.19	3.8
3.9	100.8	.06	393.2	1.23	100.1	.05	387.7	1.20	3.9
4.	101.4		405.5		100.6		399.7		4.

TABLE 23.

Based on Kutter's formula, with $n = .0225$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250 = 4.224 ft. per mile, $s = .0008$				1 in 1000 = 5.28 ft. per mile, $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	35.5	.60	14.2	.65	35.7	.60	14.3	.65	.4
.5	41.5	.52	20.7	.73	41.7	.52	20.8	.73	.5
.6	46.7	.46	28.	.79	46.9	.46	28.1	.80	.6
.7	51.3	.41	35.9	.84	51.5	.40	36.1	.83	.7
.8	55.4	.36	44.3	.88	55.5	.37	44.4	.89	.8
.9	59.	.33	53.1	.92	59.2	.33	53.3	.92	.9
1.	62.3	.30	62.3	.95	62.5	.29	62.5	.94	1.
1.1	65.3	.27	71.8	.98	65.4	.27	71.9	.98	1.1
1.2	68.	.25	81.6	1.01	68.1	.25	81.7	1.01	1.2
1.3	70.5	.23	91.7	1.02	70.6	.23	91.8	1.03	1.3
1.4	72.8	.21	101.9	1.05	72.9	.21	102.1	1.04	1.4
1.5	74.9	.19	112.4	1.05	75.	.19	112.5	1.05	1.5
1.6	76.8	.18	122.9	1.07	76.9	.18	123.	1.08	1.6
1.7	78.6	.17	133.6	1.09	78.7	.16	133.8	1.07	1.7
1.8	80.3	.16	144.5	1.11	80.3	.16	144.5	1.11	1.8
1.9	81.9	.14	155.6	1.10	81.9	.14	155.6	1.10	1.9
2.	83.3	.14	166.6	1.13	83.3	.13	166.6	1.11	2.
2.1	84.7	.12	177.9	1.11	84.6	.12	177.7	1.11	2.1
2.2	85.9	.12	189.	1.13	85.8	.13	188.8	1.15	2.2
2.3	87.1	.12	200.3	1.16	87.1	.11	200.3	1.14	2.3
2.4	88.3	.10	211.9	1.14	88.2	.10	211.7	1.13	2.4
2.5	89.3	.10	223.3	1.15	89.2	.10	223.	1.15	2.5
2.6	90.3	.10	234.8	1.17	90.2	.10	234.5	1.17	2.6
2.7	91.3	.09	246.5	1.17	91.2	.08	246.2	1.14	2.7
2.8	92.2	.08	258.2	1.15	92.	.09	257.6	1.18	2.8
2.9	93.	.08	269.7	1.17	92.9	.08	269.4	1.17	2.9
3.	93.8	.08	281.4	1.19	93.7	.08	281.1	1.19	3.
3.1	94.6	.08	293.3	1.20	94.5	.07	293.	1.16	3.1
3.2	95.4	.07	305.3	1.18	95.2	.07	304.6	1.19	3.2
3.3	96.1	.06	317.1	1.17	95.9	.06	316.5	1.16	3.3
3.4	96.7	.07	328.8	1.20	96.5	.07	328.1	1.20	3.4
3.5	97.4	.06	340.8	1.19	97.2	.06	340.1	1.18	3.5
3.6	98.	.06	352.7	1.20	97.8	.05	351.9	1.20	3.6
3.7	98.6	.05	364.7	1.20	98.3	.06	363.9	1.19	3.7
3.8	99.1	.06	376.7	1.20	98.9	.05	375.8	1.19	3.8
3.9	99.7	.05	388.7	1.20	99.4	.05	387.7	1.20	3.9
4.	100.2		400.7		99.9		399.7		4.

TABLE 24.

Based on Kutter's formula, with $n = .025$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 20000 = .264 ft. per mile				1 in 15840 = .3333 ft. per mile				$\sqrt{r}$
	$s = .00005$				$s = .000063131$				
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	23.9	.56	9.6	.48	24.8	.51	9.94	.50	.4
.5	28.9	.46	14.4	.57	29.9	.47	14.9	.57	.5
.6		.44	20.1	.64	34.6	.43	20.6	.66	.6
.7	37.9	.41	26.5	.71	38.9	.41	27.2	.72	.7
.8	42.	.38	33.6	.76	43.	.37	34.4	.77	.8
.9	45.8	.36	41.2	.82	46.7	.36	42.1	.82	.9
1.	49.4	.34	49.4	.87	50.3	.33	50.3	.87	1.
1.1	52.8	.32	58.1	.92	53.6	.32	59.	.91	1.1
1.2	56.	.31	67.3	.95	56.8	.29	68.1	.95	1.2
1.3	59.1	.29	76.8	1.	59.7	.28	77.6	.99	1.3
1.4	62.	.27	86.8	1.03	62.5	.26	87.5	1.02	1.4
1.5	64.7	.26	97.1	1.06	65.1	.25	97.7	1.05	1.5
1.6	67.3	.25	107.7	1.10	67.6	.24	108.2	1.07	1.6
1.7	69.8	.24	118.7	1.12	70.	.22	118.9	1.11	1.7
1.8	72.2	.22	129.9	1.15	72.2	.21	130.	1.12	1.8
1.9	74.4	.22	141.4	1.18	74.3	.21	141.2	1.15	1.9
2.	76.6	.21	153.2	1.20	76.4	.19	152.7	1.17	2.
2.1	78.7	.19	165.2	1.22	78.3	.18	164.4	1.19	2.1
2.2	80.6	.19	177.4	1.24	80.1	.18	176.3	1.21	2.2
2.3	82.5	.18	189.8	1.26	81.9	.17	188.4	1.22	2.3
2.4	84.3	.18	202.4	1.28	83.6	.16	200.6	1.24	2.4
2.5	86.1	.16	215.2	1.29	85.2	.15	213.	1.25	2.5
2.6	87.7	.16	228.1	1.31	86.7	.15	225.5	1.27	2.6
2.7	89.3	.16	241.2	1.33	88.2	.14	238.2	1.28	2.7
2.8	90.9	.15	254.5	1.34	89.6	.14	251.	1.29	2.8
2.9	92.4	.14	267.9	1.35	91.	.13	263.9	1.30	2.9
3.	93.8	.14	281.4	1.36	92.3	.13	276.9	1.32	3.
3.1	95.2	.13	295.	1.38	93.6	.12	290.1	1.32	3.1
3.2	96.5	.13	308.8	1.39	94.8	.12	303.3	1.33	3.2
3.3	97.8	.12	322.7	1.40	96.	.11	316.6	1.35	3.3
3.4	99.	.12	336.7	1.41	97.1	.11	330.1	1.35	3.4
3.5	100.2	.12	350.8	1.42	98.2	.10	343.6	1.36	3.5
3.6	101.4	.11	365.	1.43	99.2	.10	357.2	1.37	3.6
3.7	102.5	.11	379.3	1.43	100.2	.10	370.9	1.37	3.7
3.8	103.6	.10	393.6	1.45	101.2	.10	384.6	1.38	3.8
3.9	104.6	.11	408.1	1.47	102.2	.09	398.4	1.39	3.9
4.	105.7		422.8		103.1		412.3		4.

TABLE 24.

Based on Kutter's formula, with $n = .025$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile $s = .0001$				1 in 7500=.704 ft. per mile $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	26.5	.52	10.6	.53	27.4	.52	11.	.53	.4
.5	31.7	.47	15.9	.59	32.6	.47	16.3	.61	.5
.6	36.4	.43	21.8	.67	37.3	.43	22.4	.67	.6
.7	40.7	.40	28.5	.73	41.6	.40	29.1	.74	.7
.8	44.7	.37	35.8	.78	45.6	.36	36.5	.78	.8
.9	48.4	.34	43.6	.82	49.2	.34	44.3	.83	.9
1.	51.8	.32	51.8	.87	52.6	.31	52.6	.87	1.
1.1	55.	.30	60.5	.91	55.7	.28	61.3	.89	1.1
1.2	58.	.27	69.6	.93	58.5	.27	70.2	.94	1.2
1.3	60.7	.26	78.9	.97	61.2	.25	79.6	.96	1.3
1.4	63.3	.25	88.6	1.01	63.7	.24	89.2	1.	1.4
1.5	65.8	.22	98.7	1.01	66.1	.21	99.2	1.99	1.5
1.6	68.	.22	108.8	1.05	68.2	.21	109.1	1.04	1.6
1.7	70.2	.22	119.3	1.07	70.3	.19	119.5	1.05	1.7
1.8	72.2	.19	130.	1.08	72.2	.19	130.	1.08	1.8
1.9	74.1	.19	140.8	1.12	74.1	.17	140.8	1.08	1.9
2.	76.	.17	152.	1.12	75.8	.16	151.6	1.09	2.
2.1	77.7	.16	163.2	1.13	77.4	.16	162.5	1.13	2.1
2.2	79.3	.16	174.5	1.16	79.	.14	173.8	1.11	2.2
2.3	80.9	.15	186.1	1.17	80.4	.14	184.9	1.14	2.3
2.4	82.4	.14	197.8	1.17	81.8	.13	196.3	1.15	2.4
2.5	83.8	.13	209.5	1.18	83.1	.13	207.8	1.16	2.5
2.6	85.1	.13	221.3	1.20	84.4	.12	219.4	1.17	2.6
2.7	86.4	.12	233.3	1.20	85.6	.11	231.1	1.17	2.7
2.8	87.6	.12	245.3	1.22	86.7	.11	242.8	1.18	2.8
2.9	88.8	.11	257.5	1.22	87.8	.11	254.6	1.21	2.9
3.	89.9	.11	269.7	1.24	88.9	.10	266.7	1.20	3.
3.1	91.	.10	282.1	1.23	89 9	.09	278.7	1.19	3.1
3.2	92.	.10	294.4	1.25	90.8	.09	290.6	1.20	3.2
3.3	83.	.10	306.9	1.27	91.7	.09	302.6	1.19	3.3
3.4	94.	.09	319.6	1.25	92.5	.09	314.5	1.22	3.4
3.5	94.9	.09	332.1	1.27	93.3	.09	326.7	1.23	3.5
3.6	95.8	.08	344.8	1.27	94.2	.07	339.	1.22	3.6
3.7	96.6	.09	357.5	1.28	94.9	.08	351.2	1.24	3.7
3.8	97.5	.07	370.3	1.29	95.7	.07	363.6	1.24	3.8
3.9	98.2	.08	383.2	1.28	96.4	.07	376.	1.24	3.9
4.	99.		396.		97.1		388.4		4.

TABLE 24.

Based on Kutter's formula, with $n = .025$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile, $s = .0002$				1 in 3333.3=1.584 ft. per mile, $s = .0003$				$\sqrt{r}$ in feet
	c	diff. 01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	28.6	.53	11.4	.56	20.5	.53	11.8	.56	.4
.5	33.9	.47	17.	.62	34.8	.47	17.4	.63	.5
.6	38.6	.43	23.2	.68	39.5	.43	23.7	.70	.6
.7	42.9	.39	30.	.74	43.8	.38	30.7	.74	.7
.8	46.8	.35	37.4	.79	47.6	.35	38.1	.79	.8
.9	50.3	.33	45.3	.83	51.1	.32	46.	.83	.9
1.	53.6	.30	53.6	.87	54.3	.29	54.3	.86	1.
1.1	56.6	.29	62.3	.89	57.2	.27	62.9	.90	1.1
1.2	59.3	.26	71.2	.93	59.9	.24	71.9	.91	1.2
1.3	61.9	.24	80.5	.95	62.3	.23	81.	.94	1.3
1.4	64.3	.22	90.	.98	64.6	.21	90.4	.97	1.4
1.5	66.5	.20	99.8	.98	66.7	.20	100.1	.98	1.5
1.6	68.5	.19	109.6	1.01	68.7	.18	109.9	1.	1.6
1.7	70.4	.19	119.7	1.04	70.5	.18	119.9	1.02	1.7
1.8	72.3	.16	130.1	1.03	72.3	.17	130.1	1.03	1.8
1.9	73.9	.17	140.4	1.08	73.9	.15	140.4	1.04	1.9
2.	75.6	.15	151.2	1.07	75.4	.14	150.8	1.05	2.
2.1	77.1	.14	161.9	1.08	76.8	.13	161.3	1.05	2.1
2.2	78.5	.13	172.7	1.08	78.1	.13	171.8	1.08	2.2
2.3	79.8	.13	183.5	1.11	79.4	.12	182.6	1.08	2.3
2.4	81.1	.12	194.6	1.12	80.6	.11	193.4	1.09	2.4
2.5	82.3	.12	205.8	1.13	81.7	.11	204.3	1.10	2.5
2.6	83.5	.10	217.1	1.11	82.8	.10	215.3	1.10	2.6
2.7	84.5	.11	228.2	1.15	83.8	.10	226.3	1.11	2.7
2.8	85.6	.10	239.7	1.14	84.8	.09	237.4	1.11	2.8
2.9	86.6	.09	251.1	1.14	85.7	.09	248.5	1.13	2.9
3.	87.5	.09	262.5	1.15	86.6	.09	559.8	1.15	3.
3.1	88.4	.09	274.	1.18	87.5	.08	271.3	1.13	3.1
3.2	89.3	.08	285.8	1.15	88.3	.07	282.6	1.11	3.2
3.3	90.1	.08	297.3	1.18	89.	.08	293.7	1.16	3.3
3.4	90.9	.08	309.1	1.17	89.8	.07	305.3	1.13	3.4
3.5	91.7	.07	320.8	1.18	90.5	.06	316.6	1.15	3.5
3.6	92.4	.07	332.6	1.18	91.1	.07	328.1	1.15	3.6
3.7	93.1	.07	344.4	1.19	91.8	.06	339.6	1.16	3.7
3.8	93.8	.06	356.3	1.19	92.4	.06	351.2	1.16	3.8
3.9	94.4	.06	368.2	1.19	93.	.06	362.8	1.16	3.9
4.	95.		380.1		93.6		374.4		4.

TABLE 24.

Based on Kutter's formula, with $n = .025$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500=2.112 ft. per mile, $s = .0004$: c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 1666.7=3.168 ft. per mile, $s = .0006$: c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	30.	.53	12.	.57	30.5	.53	12.2	.57	.4
.5	35.3	.47	17.7	.63	35.8	.47	17.9	.64	.5
.6	40.	.42	24.	.69	40.5	.42	24.3	.70	.6
.7	44.2	.38	30.9	.75	44.7	.38	31.3	.75	.7
.8	48.	.33	38.4	.80	48.5	.34	38.8	.79	.8
.9	51.3	.31	46.4	.82	51.9	.31	46.7	.83	.9
1.	54.6	.29	54.6	.87	55.	.29	55.	.87	1.
1.1	57.5	.26	63.3	.88	57.9	.25	63.7	.88	1.1
1.2	60.1	.25	72.1	.93	60.4	.24	72.5	.91	1.2
1.3	62.6	.22	81.4	.93	62.8	.22	81.6	.94	1.3
1.4	64.8	.21	90.7	.97	65.	.20	91.	.95	1.4
1.5	66.9	.19	100.4	.97	67.	.19	100.5	.97	1.5
1.6	68.8	.18	110.1	.99	68.9	.17	110.2	.98	1.6
1.7	70.6	.17	120.	1.01	70.6	.17	120.	1.01	1.7
1.8	72.3	.15	130.1	1.01	72.3	.15	130.1	1.01	1.8
1.9	73.8	.15	140.2	1.04	73.8	.14	140.2	[illegible]	1.9
[illegible]	[illegible]	.14	[illegible]	1.05	[illegible]	.13	150.4	[illegible]	2.
[illegible]	[illegible]	.13	[illegible]	[illegible]	76.5	.13	160.7	[illegible]	2.1
[illegible]	[illegible]	.12	[illegible]	[illegible]	[illegible]	.12	171.2	[illegible]	2.2
[illegible]	[illegible]	.12	[illegible]	[illegible]	79.	[illegible]	181.7	[illegible]	2.3
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	.11	[illegible]	[illegible]	2.4
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE 24.

Based on Kutter's formula, with $n = .025$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250=4.224 ft. per mile $s = .0008$				1 in 1000=5.28 ft. per mile $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	30.8	.53	12.3	.58	30.9	.54	12.4	.58	.4
.5	36.1	.47	18.1	.64	36.3	.47	18.2	.64	.5
.6	40.8	.42	24.5	.70	41.	.42	24.6	.70	.6
.7	45.	.38	31.5	.75	45.2	.37	31.6	.75	.7
.8	48.8	.34	39.	.80	48.9	.34	39.1	.80	.8
.9	52.2	.30	47.	.82	52.3	.31	47.1	.83	.9
1.	55.2	.28	55.2	.86	55.4	.28	55.4	.86	1.
1.1	58.	.26	63.8	.89	58.2	.25	64.	.88	1.1
1.2	60.6	.23	72.7	.91	60.7	.23	72.8	.91	1.2
1.3	62.9	.22	81.8	.93	63.	.22	81.9	.94	1.3
1.4	65.1	.20	91.1	.96	65.2	.20	91.3	.95	1.4
1.5	67.1	.18	100.7	.95	67.2	.18	100.8	.96	1.5
1.6	68.9	.18	110.2	1.	69.	.17	110.4	.98	1.6
1.7	70.7	.16	120.2	.99	70.7	.16	120.2	.99	1.7
1.8	72.3	.15	130.1	1.01	72.3	.15	130.1	1.01	1.8
1.9	73.8	.14	140.2	1.02	73.8	.13	140.2	1.	1.9
2.	75.2	.13	150.4	1.03	75.1	.13	150.2	1.02	2.
2.1	76.5	.12	160.7	1.02	76.4	.13	160.4	1.05	2.1
2.2	77.7	.12	170.9	1.06	77.7	.11	170.9	1.03	2.2
2.3	78.9	.11	181.5	1.05	78.8	.11	181.2	1.06	2.3
2.4	80.	.10	192.	1.05	79.9	.10	191.8	1.05	2.4
2.5	81.	.10	202.5	1.07	80.9	.10	202.3	1.06	2.5
2.6	82.	.09	213.2	1.06	81.9	.09	212.9	1.07	2.6
2.7	82.9	.09	223.8	1.08	82.8	.09	223.6	1.08	2.7
2.8	83.8	.08	234.6	1.07	83.7	.08	234.4	1.07	2.8
2.9	84.6	.08	245.3	1.09	84.5	.08	245.1	1.08	2.9
3.	85.4	.08	256.2	1.10	85.3	.07	255.9	1.07	3.
3.1	86.2	.07	267.2	1.09	86.	.07	266.6	1.08	3.1
3.2	86.9	.07	278.1	1.10	86.7	.07	277.4	1.07	3.2
3.3	87.6	.07	289.1	1.11	87.4	.07	288.4	1.11	3.3
3.4	88.3	.06	300.2	1.10	88.1	.06	299.5	1.10	3.4
3.5	88.9	.06	311.2	1.10	88.7	.06	310.5	1.10	3.5
3.6	89.5	.06	322.2	1.11	89.3	.06	321.5	1.10	3.6
3.7	90.1	.05	333.3	1.12	89.9	.05	332.5	1.11	3.7
3.8	90.6	.06	344.5	1.11	90.4	.06	343.6	1.11	3.8
3.9	91.2	.05	355.6	1.12	91.	.05	354.7	1.11	3.9
4.	91.7		366.8		91.5		365.8		4.

TABLE 25.

Based on Kutter's formula, with $n = .0275$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000=.264 ft. per mile $s = .00005$				1 in 15840=.3333 ft. per mile $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	21.2	.44	8.5	.43	22.	.45	8.8	.44	.4
.5	25.6	.42	12.8	.51	26.5	.42	13.2	.52	.5
.6	29.8	.40	17.9	.58	30.7	.40	18.4	.59	.6
.7	33.8	.37	23.7	.63	34.7	.37	24.3	.64	.7
.8	37.5	.35	30.	.69	38.4	.35	30.7	.70	.8
.9	41.	.34	36.9	.75	41.9	.32	37.7	.74	.9
1.	44.4	.31	44.4	.78	45.1	.31	45.1	.79	1.
1.1	47.5	.30	52.2	.84	48.2	.29	53.	.83	1.1
1.2	50.5	.28	60.6	.87	51.1	.28	61.3	.87	1.2
1.3	53.3	.27	69.3	.91	53.9	.26	70.	.90	1.3
1.4	56.	.26	78.4	.95	56.5	.24	79.	.94	1.4
1.5	58.6	.24	87.9	.98	58.9	.24	88.4	.96	1.5
1.6	61.	.24	97.7	1.01	61.3	.22	98.	1.00	1.6
1.7	63.4	.22	107.8	1.05	63.5	.21	108.	1.01	1.7
1.8	65.6	.22	118.3	1.04	65.6	.20	118.1	1.03	1.8
1.9	67.8	.20	128.7	1.09	67.6	.20	128.4	1.06	1.9
2.	69.8	.19	139.6	1.11	69.5	.19	139.	1.10	2.
2.1	71.7	.19	150.7	1.13	71.4	.18	150.	1.10	2.1
2.2	73.6	.18	162.	1.15	73.2	.17	161.	1.12	2.2
2.3	75.4	.18	173.5	1.17	74.9	.16	172.2	1.14	2.3
2.4	77.2	.16	185.2	1.19	76.5	.15	183.6	1.15	2.4
2.5	78.8	.16	197.1	1.20	78.	.15	195.1	1.17	2.5
2.6	80.4	.16	209.1	1.22	79.5	.15	206.8	1.18	2.6
2.7	82.	.15	221.3	1.24	81.	.13	218.6	1.19	2.7
2.8	83.5	.14	233.7	1.25	82.3	.14	230.5	1.21	2.8
2.9	84.9	.14	246.2	1.27	83.7	.12	242.6	1.22	2.9
3.	86.3	.13	258.9	1.28	84.9	.12	254.8	1.23	3.
3.1	87.6	.13	271.7	1.28	86.1	.12	267.1	1.24	3.1
3.2	88.9	.13	284.5	1.31	87.3	.12	279.5	1.25	3.2
3.3	90.2	.12	297.6	1.31	88.5	.11	292.	1.25	3.3
3.4	91.4	.11	310.7	1.32	89.6	.10	304.5	1.27	3.4
3.5	92.5	.12	323.9	1.33	90.6	.10	317.2	1.28	3.5
3.6	93.7	.11	337.2	1.34	91.6	.11	330.	1.28	3.6
3.7	94.8	.10	350.6	1.35	92.7	.09	342.8	1.29	3.7
3.8	95.8	.11	364.1	1.37	93.6	.09	355.7	1.30	3.8
3.9	96.9	.10	377.8	1.38	94.5	.09	368.7	1.31	3.9
4.	97.9		391.6	1.38	95.4		381.8		4.

TABLE 25.

Based on Kutter's formula, with $n = .0275$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile, $s = .0001$				1 in 7500=.704 ft. per mile, $s = .000133333$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	23.4	.46	9.4	.46	24.2	.47	9.7	.47	.4
.5	28.	.43	14.	.54	28.9	.43	14.4	.55	.5
.6	32.3	.40	19.4	.60	33.2	.39	19.9	.61	.6
.7	36.3	.36	25.4	.65	37.1	.36	26.	.66	.7
.8	39.9	.34	31.9	.71	40.7	.34	32.6	.71	.8
.9	43.2	.32	39.	.75	44.1	.31	39.7	.75	.9
1.	46.5	.29	46.5	.79	47.2	.29	47.2	.79	1.
1.1	49.4	.28	54.4	.82	50.1	.26	55.1	.82	1.1
1.2	52.2	.26	62.6	.86	52.7	.25	63.3	.85	1.2
1.3	54.8	.24	71.2	.89	55.2	.24	71.8	.88	1.3
1.4	57.2	.23	80.1	.92	57.6	.22	80.6	.91	1.4
1.5	59.5	.22	89.3	.94	59.8	.21	89.7	.93	1.5
1.6	61.7	.20	98.7	.96	61.9	.19	99.	.94	1.6
1.7	63.7	.19	108.3	.99	63.8	.19	108.4	.98	1.7
1.8	65.6	.19	118.2	1.	65.7	.17	118.2	.98	1.8
1.9	67.5	.17	128.2	1.02	67.4	.16	128.	1.01	1.9
2.	69.2	.17	138.4	1.04	69.	.16	138.1	1.01	2.
2.1	70.9	.15	148.8	1.04	70.6	.15	148.2	1.03	2.1
2.2	72.4	.15	159.4	1.07	72.1	.14	158.5	1.05	2.2
2.3	73.9	.15	170.1	1.08	73.5	.13	169.	1.05	2.3
2.4	75.4	.13	180.9	1.09	74.8	.13	179.5	1.07	2.4
2.5	76.7	.13	191.8	1.10	76.1	.12	190.2	1.08	2.5
2.6	78.	.13	202.8	1.12	77.3	.12	201.	1.10	2.6
2.7	79.3	.12	214.	1.13	78.5	.11	212.	1.08	2.7
2.8	80.5	.11	225.3	1.14	79.6	.10	222.8	1.09	2.8
2.9	81.6	.11	236.7	1.14	80.6	.10	233.7	1.12	2.9
3.	82.7	.11	248.1	1.15	81.6	.10	244.9	1.12	3.
3.1	83.8	.10	259.6	1.17	82.6	.09	256.1	1.12	3.1
3.2	84.8	.10	271.3	1.17	83.5	.09	267.3	1.14	3.2
3.3	85.8	.09	283.	1.17	84.4	.09	278.7	1.13	3.3
3.4	86.7	.09	294.7	1.19	85.3	.08	290.	4.15	3.4
3.5	87.6	.09	306.6	1.19	86.1	.08	301.5	1.14	3.5
3.6	88.5	.08	318.5	1.19	86.9	.08	312.9	1.16	3.6
3.7	89.3	.08	330.4	1.20	87.7	.07	224.5	1.16	3.7
3.8	90.1	.08	342.4	1.21	88.4	.07	336.1	1.16	3.8
3.9	90.9	.07	354.5	1.21	89.1	.07	347.7	1.17	3.9
4.	91.6		366.6		89.8		359.4		4.

TABLE 25.

Based on Kutter's formula, with $n = .0275$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile $s = .0002$ c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 3333.3=1.584 ft. per mile $s = .0003$ c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	25.2	.47	10.1	.49	25.9	.48	10.4	.50	.4
.5	29.9	.43	15.	.55	30.7	.43	15.4	.56	5
.6	34.2	.39	20.5	.62	35.	.39	21.	.62	.6
.7	38.1	.36	26.7	.67	38.9	.35	27.2	.67	.7
.8	41.7	.33	33.4	.71	42.4	.33	33.9	.72	.8
.9	45.	.30	40.5	.75	45.7	.29	41.1	.75	.9
1.	48.	.28	48.	.79	48.6	.28	48.6	.79	1
1.1	50.8	.26	55.9	.82	51.4	.25	56.5	.82	1.1
1.2	53.4	.24	64.1	.84	53.9	.23	64.7	.84	1.2
1.3	55.8	.22	72.5	.87	56.2	.22	73.1	.86	1.3
1.4	58.	.21	81.2	.90	58.4	.20	81.7	.89	1.4
1.5	60.1	.20	90.2	.92	60.4	.19	90.6	.90	1.5
1.6	62.1	.18	99.4	.93	62.3	.17	99.6	.92	1.6
1.7	63.9	.18	108.7	.95	64.	.17	108.8	.94	1.7
1.8	65.7	.16	118.2	.97	65.7	.15	118.2	.95	1.8
1.9	67.3	.15	127.9	.98	67.2	.15	127.7	.97	1.9
2.	68.8	.15	137.7	.99	68.7	.14	137.4	.97	2.
2.1	70.3	.13	147.6	1.00	70.1	.13	147.1	.99	2.1
2.2	71.6	.13	157.6	1.02	71.4	.12	157.	.99	2.2
2.3	72.9	.13	167.8	1.03	72.6	.11	166.9	1.01	2.3
2.4	74.2	.12	178.1	1.03	73.7	.11	177.	1.01	2.4
2.5	75.4	.11	188.4	1.05	74.8	.11	187.1	1.02	2.5
2.6	76.5	.10	198.9	1.05	75.9	.10	197.3	1.03	2.6
2.7	77.5	.11	209.4	1.06	76.9	.10	207.6	1.04	2.7
2.8	78.6	.09	220.	1.06	77.9	.09	218.	1.04	2.8
2.9	79.5	.09	230.6	1.08	78.8	.08	228.4	1.05	2.9
3.	80.4	.09	241.4	1.08	79.6	.08	238.9	1.05	3.
3.1	81.3	.09	252.2	1.08	80.4	.08	249.4	1.06	3.1
3.2	82.2	.08	263.	1.09	81.2	.08	260.	1.06	3.2
3.3	83.	.08	273.9	1.10	82.	.07	270.6	1.07	3.3
3.4	83.8	.07	284.9	1.10	82.7	.07	281.3	1.07	3.4
3.5	84.5	.08	295.9	1.10	83.4	.07	292.	1.07	3.5
3.6	85.3	.06	306.9	1.11	84.1	.06	302.7	1.08	3.6
3.7	85.9	.07	318.	1.11	84.7	.06	313.5	1.08	3.7
3.8	86.6	.07	329.1	1.12	85.3	.06	324.3	1.09	3.8
3.9	87.3	.06	340.3	1.12	85.9	.06	335.2	1.08	3.9
4.	87.9		351.5		86.5		346.		4.

TABLE 25.

Based on Kutter's formula, with $n = .0275$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 2500=2.114 ft. per mile				1 in 1666.7=3.168 ft. per mile				$\sqrt{r}$
	$s = .0004$				$s = .0006$				
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	26.4	.48	10.5	.51	26.8	.48	10.7	.51	.4
.5	31.2	.43	15.6	.57	31.6	.43	15.8	.57	.5
.6	35.5	.38	21.3	.62	35.9	.39	21.5	.63	.6
.7	39.3	.35	27.5	.68	39.8	.35	27.8	.68	.7
.8	42.8	.32	34.3	.71	43.3	.31	34.6	.72	.8
.9	46.	.30	41.4	.76	46.4	.29	41.8	.75	.9
1.	49.	.27	49.	.78	49.3	.27	49.3	.79	1.
1.1	51.7	.24	56.8	.81	52.	.24	57.2	.81	1.1
1.2	54.1	.23	64.9	.85	54.4	.23	65.3	.84	1.2
1.3	56.4	.21	73.4	.86	56.7	.20	73.7	.85	1.3
1.4	58.5	.20	82.	.88	58.7	.20	82.2	.88	1.4
1.5	60.5	.19	90.8	.90	60.7	.18	91.	.89	1.5
1.6	62.4	.17	99.8	.91	62.5	.16	99.9	.91	1.6
1.7	64.1	.16	108.9	.93	64.1	.16	109.	.92	1.7
1.8	65.7	.15	118.2	.95	65.7	.15	118.2	.94	1.8
1.9	67.2	.14	127.7	.95	67.2	.13	127.6	.94	1.9
2.	68.6	.13	137.2	.97	68.5	.13	137.	.96	2.
2.1	69.9	.12	146.9	.96	69.8	.12	146.6	.97	2.1
2.2	71.1	.13	156.5	1.	71.	.12	156.3	.97	2.2
2.3	72.4	.12	166.5	1.	72.2	.11	166.	.99	2.3
2.4	73.6	.10	176.5	1.	73.3	.10	175.9	.99	2.4
2.5	74.6	.10	186.5	1.	74.3	.10	185.8	.99	2.5
2.6	75.6	.10	196.5	1.02	75.3	.09	195.7	1.01	2.6
2.7	76.6	.10	206.7	1.04	76.2	.09	205.8	1.01	2.7
2.8	77.6	.08	217.1	1.03	77.1	.08	215.9	1.01	2.8
2.9	78.4	.08	227.4	1.02	77.9	.08	226.	1.02	2.9
3.	79.2	.08	237.6	1.03	78.7	.08	236.2	1.02	3.
3.1	80.	.07	247.9	1.05	79.5	.07	246.4	1.03	3.1
3.2	80.7	.08	258.4	1.04	80.2	.07	256.7	1.03	3.2
3.3	31.5	.07	268.8	1.06	80.9	.07	267.	1.04	3.3
3.4	82.2	.06	279.4	1.05	81.6	.06	277.4	1.04	3.4
3.5	82.8	.07	289.9	1.06	82.2	.06	287.8	1.04	3.5
3.6	83.5	.06	300.5	1.06	82.8	.06	298.2	1.05	3.6
3.7	84.1	.06	311.1	1.07	83.4	.06	308.7	1.05	3.7
3.8	84.7	.05	321.8	1.07	84.	.05	319.2	1.05	3.8
3.9	85.2	.06	332.5	1.07	84.5	.06	329.7	1.06	3.9
4.	85.8		343.2		85.1		340.3		4.

TABLE 25.

Based on Kutter's formula, with $n = .0275$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250=4.224 ft. per mile $s = .0008$				1 in 1000=5.28 ft. per mile $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	27.1	48	10.8	.51	27.2	.48	10.9	.51	.4
.5	31.9	.43	15.9	.58	32.	.43	16.	.58	.5
.6	36.2	.38	21.7	.63	36.3	.39	21.8	.63	.6
.7	40.	.35	28.	.68	40.2	.34	28.1	.68	.7
.8	43.5	.32	34.8	.72	43 6	.32	34.9	.72	.8
.9	46.7	.28	42.	.75	46.8	.28	42.1	.75	.9
1.	49.5	.27	49.5	.78	49.6	.27	49.6	.79	1.
1.1	52.2	.24	57.3	.82	52.3	.23	57.5	.81	1.1
1.2	54.6	.22	65.5	.83	54.6	.23	65.6	.83	1.2
1.3	56.8	20	73.8	.86	56.9	.20	73.9	.85	1.3
1.4	58.8	.19	82.4	.87	58.9	.19	82.4	.88	1.4
1.5	60.7	.18	91.1	.89	60.8	.17	91.2	.89	1.5
1.6	62.5	.17	100.	.91	62.5	.17	100.1	.90	1.6
1.7	64.2	.15	109.1	.91	64.2	.15	109.1	.91	1.7
1.8	65.7	.14	118.2	.94	65.7	.14	118.2	.93	1.8
1.9	67.1	.14	127 6	.94	67.1	.14	127.5	.94	1.9
2.	68.5	.13	137.	.95	68.5	.12	136.9	.95	2.
2.1	69.8	.11	146.5	.96	69.7	.12	146.4	.96	2.1
2.2	70.9	.12	156.1	.97	70.9	.11	156.	.96	2.2
2.3	72.1	.11	165.8	.98	72.	.11	165.6	.98	2.3
2.4	73.2	.10	175.6	.98	73.1	.10	175.4	.98	2.4
2.5	74.2	.09	185.4	.99	74.1	.09	185.2	.99	2.5
2.6	75.1	.09	195.3	1.	75.	.09	195.1	.99	2.6
2.7	76.	.09	205.3	1.	75.9	.09	205.	1.	2.7
2.8	76.9	.08	215.3	1.01	76.8	.08	215.	1.	2.8
2.9	77.7	.08	225.4	1.01	77.6	.08	225.	1.01	2.9
3.	78.5	.07	235.5	1.02	78.4	.07	235.1	1.01	3.
3.1	79.2	.07	245.5	1 02	79.1	.07	245.2	1.02	3.1
3.2	79.9	.08	255.9	1.02	79.8	.07	255.4	1.02	3.2
3.3	80.7	.06	266.1	1.03	80.5	.06	265.6	1.02	3.3
3.4	81.3	.06	276 4	1.03	81.1	.07	275.8	1.03	3.4
3.5	81.9	.06	286.7	1.04	81.8	.05	286.1	1.03	3.5
3.6	82.5	06	297.1	1.04	82.3	.06	296.4	1.04	3.6
3.7	83.1	.06	307.5	1.04	82.9	.05	306.8	1.03	3.7
3.8	83.7	.05	317.9	1.04	83.4	.06	317.1	1.04	3.8
3.9	84.2	.05	328.3	1.05	84.	.05	327.5	1.04	3.9
4.	84.7		338.8		84.5		337.9		4.

TABLE 26.

Based on Kutter's formula, with $n = .030$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000 = .264 ft. per mile, $s = .00005$				1 in 15840 = .3333 ft. per mile, $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	19.	.40	7.59	.39	19.6	.42	7.86	.40	.4
.5	23.	.38	11.5	.46	23 8	.38	11.9	.47	5
.6	26.8	.37	16.1	.52	27.6	.36	16.6	.53	6
.7	30.5	.34	21.3	.58	31.2	.34	21.9	.58	.7
.8	33.9	.32	27.1	.63	34.6	33	27.7	.64	8
.9	37.1	.31	33.4	.68	37.9	.30	34.1	.68	.9
1.	40.2	.29	40.2	.72	40.9	.28	40.9	.72	1.
1.1	43.1	.28	47.4	.77	43.7	.28	48.1	.77	1.1
1.2	45.9	.27	55.1	.80	46.5	.25	55.8	.80	1.2
1.3	48.6	.25	63.1	.84	49.	.24	63.8	.82	1.3
1.4	51.1	.24	71.5	.88	51.4	.24	72.	.87	1.4
1.5	53.5	.23	80.3	.90	53.8	.22	80.7	.89	1.5
1.6	55.8	.22	89.3	.93	56.	.21	89.6	.92	1.6
1.7	58.	.21	98.6	.96	58.1	.21	98.8	.95	1.7
1.8	60.1	.21	108.2	.99	60.2	.19	108.3	.96	1.8
1.9	62.2	.19	118.1	1.01	62.1	.18	117.9	.99	1.9
2.	64.1	.19	128.2	1.03	63.9	.18	127.8	1.01	2.
2.1	66.	.18	138.5	1.06	65.7	.17	137.9	1.03	2.1
2.2	67.8	.17	149.1	1.06	67.4	.16	148.2	1.05	2.2
2.3	69.5	.17	159.9	1.08	69.	.15	158.7	1.06	2 3
2.4	71.2	.16	170.8	1.11	70.5	.15	169.3	1.10	2.4
2.5	72.8	.15	181.9	1.13	72.	.16	180.3	1.10	2.5
2.6	74.3	.14	193.2	1.12	73.6	.13	191.3	1.08	2 6
2.7	75.7	.15	204.4	1.19	74.9	.13	202.1	1.12	2.7
2.8	77.2	.14	216.3	1.17	76.2	.13	213.3	1.13	2 8
2.9	78.6	.14	228.	1.19	77.5	.12	224.6	1.15	2.9
3.	80.	.13	239.9	1.20	78.7	.12	236.1	1.16	3.
3.1	81.3	.12	251.9	1.21	79.9	.11	247.7	1.16	3.1
3.2	82.5	.12	264.	1.23	81.	.12	259.3	1.18	3.2
3.3	83 7	.12	276.3	1.24	82 2	.10	271.1	1.19	3.3
3.4	84.9	.11	288.7	1 24	83.2	.11	283.	1.20	3.4
3.5	86.	.11	301.1	1.26	84.3	.10	295.	1.20	3.5
3.6	87.1	.11	313 7	1.27	85.3	.10	307.	1.21	3.6
3.7	88.2	.11	326.4	1.28	86.3	.09	319.1	1.22	3.7
3.8	89.3	.10	339.2	1.28	87.2	09	331.3	1.23	3.8
3.9	90.3	.09	352.	1.30	88.1	.09	343.6	1.24	3.9
4.	91.2		365.		89.		356.		4.

TABLE 26:

Based on Kutter's formula, with $n = .030$.

$c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} =$$

All slopes greater than 1 in 1000 have the sam

$\sqrt{r}$ in feet	1 in 10000=.528 ft. per mile $s = .0001$				1 in 7500 = $s =$	
	c	diff .01	$c\sqrt{r}$	diff. .01	c	d .(
.4	20.9	.42	8.4	.42	21.5	.4
.5	25.1	.39	12.6	.48	25.8	.3
.6	29.	.36	17.4	.54	29.7	.3
.7	32.6	.34	22.8	.60	33.3	.3
.8	36	.31	28.8	.64	36.7	.3
.9	39.1	.30	35.2	.69	39.8	.2
1.	42.1	.27	42.1	.72	42.7	.2
1.1	44.8	.26	49.3	.76	45.4	.2
1.2	47.4	.25	56.9	.80	47.9	.2
1.3	49.9	.23	64.9	.82	50.3	.2
1.4	52.2	.21	73.1	.84	52.5	.2
1.5	54.3	.21	81.5	.87	54.6	.1
1.6	56.4	.19	90.2	.89	56.5	.1
1.7	58.3	.19	99.1	.93	58.4	.1
1.8	60.2	.17	108.4	.92	60.2	.1
1.9	61.9	.17	117.6	.96	61.9	.1
2.	63.6	.16	127.2	.97	63.4	.1
2.1	65.2	.15	136.9	.98	64.9	.1
2.2	66.7	.14	146.7	.99	66.4	.1
2.3	68.1	.14	156.6	1.02	67.7	.1
2.4	69.5	.13	166.8	1.02	69.	.1
2.5	70.8	.13	177.	1.05	70.3	.1
2.6	72.1	.12	187.5	1.04	71.5	.1
2.7	73.3	.12	197.9	1.07	72.6	.1
2.8	74.5	.11	208.6	1.06	73.7	.1
2.9	75.6	.10	219.2	1.06	74.7	.1
3.	76.6	.11	229.8	1.11	75.7	.1
3.1	77.7	.10	240.9	1.09	76.7	.0
3.2	78.7	.09	251.8	1.09	77.6	.0
3.3	79.6	.09	262.7	1.10	78.4	.0
3.4	80.5	.09	273.7	1.13	79.2	.0
3.5	81.4	.09	285.	1.12	80.	.0
3.6	82.3	.08	296.2	1.13	80.8	.0
3.7	83.1	.08	307.5	1.13	81.6	.0
3.8	83.9	.08	318.8	1.14	82.3	.0
3.9	84.7	.07	330.2	1.15	83.	.0
4.	85.4		341.7		83.7	

TABLE 26.

Based on Kutter's formula, with $n = .030$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000 = 1.056 ft. per mile, $s = .0002$				1 in 3333.3 = 1.584 ft. per mile, $s = .0003$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	22.4	.43	8.96	.45	23.1	.43	9.24	.45	.4
.5	26.7	.42	13.4	.50	27.4	.40	13.7	.51	.5
.6	30.7	.36	18.4	.56	31.4	.36	18.8	.57	.6
.7	34.3	.33	24.	.61	35.	.32	24.5	.61	.7
.8	37.6	.30	30.1	.64	38.2	.30	30.6	.65	.8
.9	40.6	.29	36.5	.70	41.2	.28	37.1	.69	.9
1.	43.5	.26	43.5	.72	44.	.26	44.	.73	1.
1.1	46.1	.24	50.7	.75	46.6	.24	51.3	.75	1.1
1.2	48.5	.23	58.2	.78	49.	.22	58.8	.78	1.2
1.3	50.8	.21	66.	.81	51.2	.20	66.6	.79	1.3
1.4	52.9	.20	74.1	.83	53.2	.19	74.5	.82	1.4
1.5	54.9	.19	82.4	.85	55.1	.18	82.7	.83	1.5
1.6	56.8	.17	90.9	.86	56.9	.17	91.	.86	1.6
1.7	58.5	.17	99.5	89	58.6	.16	99.6	.88	1.7
1.8	60.2	.16	108.4	.90	60 2	.15	108.4	.88	1.8
1.9	61.8	.14	117.4	.90	61.7	.14	117.2	.90	1.9
2.	63.2	.14	126.4	.93	63.1	.13	126.2	.90	2.
2.1	64.6	14	135.7	.95	64.4	.13	135.2	.93	2.1
2.2	66	.12	145.2	.94	65.7	.12	144.5	.94	2.2
2.3	67.2	.12	154.6	.96	66.9	.11	153.9	.93	2.3
2.4	68.4	.12	164.2	.98	68.	.11	163.2	.96	2.4
2.5	69.6	.11	174	.98	69.1	.10	172.8	.95	2.5
2.6	70.7	.10	183.8	.98	70.1	.10	182.3	.97	2.6
2.7	71.7	.10	193.6	1.	71.1	.09	192.	.96	2.7
2.8	72.7	.09	203.6	.98	72.	.09	201.6	.98	2.8
2.9	73.6	.09	213.4	1.01	72.9	.08	211 4	.97	2.9
3.	74.5	.09	223.5	1.02	73.7	.09	221.1	1.02	3.
3.1	75.4	.08	233.7	1.01	74.6	.09	231.3	.97	3.1
3.2	76.2	.08	243.8	1.03	75.3	.08	241.	1.01	3.2
3.3	77.	.08	254.1	1.03	76.1	.07	251.1	1.01	3.3
3.4	77.8	.08	264.5	1.04	76.8	.07	261.1	1.	3.4
3.5	78.6	.07	274.9	1.04	77.5	.06	271.2	1.01	3.5
3.6	79.3	.06	285 3	1.05	78.1	.07	281.3	1.01	3.6
3.7	79.9	.07	295.8	1.05	78.8	.06	291.5	1.02	3.7
3.8	80.6	.06	306.3	1.05	79.4	.06	301.7	1.02	3.8
3.9	81.2	.07	316.8	1.06	80.	.06	311.9	1.02	3.9
4.	81.9		327.4		80.6		322.2		4.

TABLE 26.

Based on Kutter's formula, with $n = .030$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 2500 = 2.114 ft. per mile, $s = .0004$				1 in 1666.7 = 3.168 ft. per mile, $s = .0006$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	23.5	.43	9.4	.45	23 9	.43	9.6	.45	.4
.5	27.8	.40	13.9	.52	28.2	.40	14.1	.52	.5
.6	31.8	.35	19.1	.56	32.2	.36	19.3	.58	.6
.7	35.3	.33	24.7	.62	35.8	.32	25.1	.61	.7
.8	38.6	.30	30.9	.65	39.	.29	31.2	.65	.8
.9	41.6	.27	37.4	.69	41.9	.28	37.7	.70	.9
1.	44.3	.25	44.3	.72	44.7	.24	44.7	.71	1.
1.1	46.8	.24	51.5	.75	47.1	.23	51.8	.75	1.1
1.2	49.2	.22	59.	.78	49.4	.22	59 3	.78	1.2
1.3	51.4	.20	66.8	.80	51.6	.19	67.1	.78	1.3
1.4	53.4	.18	74.8	.80	53.5	.19	74.9	.82	1.4
1.5	55.2	.18	82.8	.84	55 4	.17	83.1	.83	1.5
1.6	57.	.17	91.2	.86	57.1	.16	91.4	.84	1.6
1.7	58.7	.15	99.8	.86	58.7	.16	99.8	.86	1.7
1.8	60.2	.14	108.4	.86	60.2	.14	108.4	.86	1.8
1.9	61.6	.14	117.	.90	61.6	.13	117.	.88	1.9
2.	63.	.13	126.	.90	62.9	.13	125.8	.90	2.
2.1	64 3	.12	135.	.91	64.2	.12	134.8	.91	2.1
2.2	65.5	.12	144.1	.93	65.4	.11	143.9	.91	2.2
2.3	66.7	.11	153.4	.93	66.5	.11	153.	.94	2.3
2.4	67.8	.10	162.7	.93	67.6	.10	162.4	.91	2.4
2.5	68.8	.10	172.	.95	68.6	.09	171.5	.92	2.5
2.6	69.8	.10	181.5	.97	69.5	.09	180.7	.94	2.6
2.7	70.8	.09	191.2	.96	70.4	.09	190.1	.95	2.7
2.8	71.7	.08	200.8	.95	71.3	.08	199.6	.95	2.8
2.9	72 5	.08	210.3	.96	72.1	.08	209.1	.96	2.9
3.	73 3	.08	219.9	1.	72.9	.08	218.7	.98	3.
3.1	74.1	.08	229.9	.98	73.7	.07	228.5	.96	3.1
3.2	74.9	.07	239.7	.98	74.4	.07	238.1	.97	3.2
3.3	75.6	.07	249.5	.99	75 1	.06	247.8	.96	3.3
3 4	76.3	.06	259.4	.99	75.7	.07	257.4	.99	3.4
3.5	76.9	.07	269.3	.99	76.4	.06	267.3	.98	3.5
3.6	77.6	.06	279.2	1.01	77.	.06	277.1	.98	3.6
3.7	78 2	.06	289.3	1.	77.6	.05	286.9	.98	3.7
3.8	78.8	.06	299.3	1.01	78.1	.06	296.8	.99	3.8
3.9	79 3	.05	309 4	1.01	78 7	.05	306.7	.99	3.9
4	79.9	.06	319.5		79.2		316.7		4.

TABLE 26.

Based on Kutter's formula, with $n = .030$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250=4.224 ft. per mile $s = .0008$				1 in 1000=5.28 ft. per mile $s = .001$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	24.1	.44	9.6	.47	24.2	.44	9.7	.46	.4
.5	28.5	.39	14.3	.53	28.6	.39	14.3	.52	.5
.6	32.4	.36	19.4	.58	32.5	.36	19.5	.58	.6
.7	36.	.32	25.2	.62	36.1	.32	25.3	.61	.7
.8	39.2	.29	31.4	.65	39.3	.29	31.4	.66	.8
.9	42.1	.27	37.9	.69	42.2	.27	38.	.69	.9
1.	44.8	.25	44.8	.72	44.9	.25	44.9	.72	1.
1.1	47.3	.23	52.	.75	47.4	.23	52.1	.75	1.1
1.2	49.6	.21	59.5	.77	49.7	.21	59.6	.77	1.2
1.3	51.7	.19	67.2	.78	51.8	.19	67.3	.79	1.3
1.4	53.6	.18	75.	.81	53.7	.18	75.2	81	1.4
1.5	55.4	.17	83.1	.83	55.5	.17	83.3	.82	1.5
1.6	57.1	.16	91.4	.84	57.2	.15	91.5	.83	1.6
1.7	58.7	.15	99.8	.86	58.7	.15	99.8	.86	1.7
1.8	60.2	.14	108.4	.86	60.2	.14	108.4	.86	1.8
1.9	61.6	.13	117.	.88	61.6	.13	117.	.88	1.9
2.	62.9	.12	125.8	.88	62.9	.12	125.8	.88	2.
2.1	64.1	.12	134.6	.91	64.1	.12	134.6	.89	2.1
2.2	65.3	.11	143.7	.90	65.3	.10	143.7	.88	2.2
2.3	66.4	.10	152.7	.91	66.3	.11	152.5	.93	2.3
2.4	67.4	.10	161.8	.92	67.4	.09	161.8	.90	2.4
2.5	68.4	.10	171.	.94	68.3	.10	170.8	.94	2.5
2.6	69.4	.09	180.4	.94	69.3	.09	180.2	.93	2.6
2.7	70.3	.08	189.8	.93	70.2	.08	189.5	.93	2.7
2.8	71.1	.08	199.1	.94	71.	.08	198.8	94	2.8
2.9	71.9	.08	208.5	.96	71.8	.08	208.2	96	2.9
3.	72.7	.07	218.1	.94	72.6	.07	217.8	.94	3.
3.1	73.4	.07	227.5	.96	73.3	.07	227.2	.96	3.1
3.2	74.1	.07	237.1	.97	74	.06	236.8	.94	3.2
3.3	74.8	.06	246.8	.96	74.6	.07	246.2	.98	3.3
3.4	75.4	.07	256.4	.98	75.3	.06	256.	.97	3.4
3.5	76.1	.06	266.2	.98	75.9	.06	265.7	.96	3.5
3.6	76.7	.05	276.	.98	76.5	.05	275.3	.98	3.6
3.7	77.2	.06	285.8	.98	77.	.06	285.1	98	3.7
3.8	77.8	.05	295.6	.98	77.6	.05	294.8	.98	3.8
3.9	78.3	.05	305.4	.98	78.1	.05	304.6	.98	3.9
4.	78.8	.05	315.2		78.6	.05	314.4		4

TABLE 27.

Based on Kutter's formula, with $n = .035$. Values of the factors c and $c\sqrt{r}$ for use in the formula

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 20000 = .264 ft. per mile, $s = .00005$				1 in 15840 = .3333 ft. per mile, $s = .000063131$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	15.6	.34	6.3	.32	16.2	.34	6.46	.34	.4
.5	19.	.33	9.5	.39	19.6	.33	9.8	.39	.5
.6	22.3	.31	13.4	.44	22.9	.31	13.7	.45	.6
.7	25.4	.29	17.8	.49	26.	.29	18.2	.49	.7
.8	28.3	.28	22.7	.53	28.9	.28	23.1	.54	.8
.9	31.1	.27	28.	.58	31.7	.26	28.5	.58	.9
1.	33.8	.26	33.8	.62	34.3	.26	34.3	.63	1.
1.1	36.4	.24	40.	.66	36.9	.24	40.6	.65	1.1
1.2	38.8	.23	46.6	.69	39.3	.22	47.1	.69	1.2
1.3	41.1	.23	53.5	.73	41.5	.22	54	.72	1.3
1.4	43.4	.22	60.8	.75	43.7	.21	61.2	.75	1.4
1.5	45.6	.20	68.3	.79	45.8	.20	68.7	.78	1.5
1.6	47.6	.20	76.2	.81	47.8	.19	76.5	.80	1.6
1.7	49.6	.19	84.3	.84	49.7	.19	84.5	.83	1.7
1.8	51.5	.19	92.7	.87	51.6	.17	92.8	.85	1.8
1.9	53.4	.17	101.4	.88	53.3	.17	10[illegible].3	.87	1.9
2.	[illegible]	[illegible]	110.2	.92	55.	[illegible]	[illegible]	[illegible]	2.
2.1	[illegible]	[illegible]	119.4	.93	56.6	[illegible]	[illegible]	.91	2.1
2.2	[illegible]	[illegible]	[illegible]	.95	58.2	[illegible]	[illegible]	[illegible]	2.2
2.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.3
2.4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.4
2.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.5
2.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.6
2.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.7
2.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.8
2.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.9
3.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.
3.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.1
3.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.2
3.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.3
3.4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.4
3.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.5
3.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.6
3.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.7
3.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.8
3.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.9
4.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.

TABLE 27.

Based on Kutter's formula, with $n = .035$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 10000=0.528 ft. per mile, $s = .0001$				1 in 7500=0.704 ft. per mile, $s = .000133333.$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	17.1	.36	6.84	.36	17.6	.36	7.04	.36	.4
.5	20.7	.33	10.4	.40	21.2	.34	10.6	.42	.5
.6	24.	.31	14.4	.46	24.6	.31	14.8	.46	.6
.7	27.1	.29	19.	.50	27.7	.29	19.4	.51	.7
.8	30.	.27	24.	.54	30.6	.27	24.5	.55	.8
.9	32.7	.26	29.4	.59	33.3	.25	30.	.58	.9
1.	35.3	.24	35.3	.62	35.8	.24	35.8	.62	1.
1.1	37.7	.23	41.5	.65	38.2	.23	42.	.66	1.1
1.2	40.	.22	48.	.69	40.5	.21	48.6	.68	1.2
1.3	42.2	.21	54.9	.71	42.6	.20	55.4	.70	1.3
1.4	44.3	.19	62.	.73	44.6	.19	62.4	.74	1.4
1.5	46.2	.19	69.3	.77	46.5	.18	69.8	.75	1.5
1.6	48.1	.18	77.	.78	48.3	17	77.3	.77	1.6
1.7	49.9	.17	84.8	.81	50.	.16	85.	.79	1.7
1.8	51.6	.16	92.9	.82	51.6	.15	92.9	80	1.8
1.9	53.2	.15	101.1	.83	53.1	.15	100.9	.80	1.9
2.	54.7	.15	109.4	.86	54.6	.14	109.2	.83	2.
2.1	56.2	.14	118.	.87	56.	.13	117.6	.84	2.1
2.2	57.6	.13	126.7	.88	57.3	13	126.1	.85	2.2
2.3	58.9	.13	135.5	.90	58.6	.12	134.8	87	2.3
2.4	60.2	.13	144.5	.93	59.8	.11	143.5	.87	2.4
2.5	61.5	.12	153.8	.92	60.9	.12	152.3	.88	2.5
2.6	62.7	.11	163.	.93	62.1	.10	161.5	.89	2.6
2.7	63.8	.11	172.3	.94	63.1	11	170.4	.94	2.7
2.8	64.9	.11	181.7	.97	64.2	.09	179.8	90	2.8
2.9	66.	10	191.4	.96	65.1	.10	188.8	.95	2.9
3.	67.	.10	201.	.98	66.1	.09	198.3	.94	3.
3.1	68.	09	210.8	.97	67.	.09	207.7	.96	3.1
3.2	68.9	.09	220.5	.98	67.9	.08	217.3	.94	3.2
3.3	69.8	.09	230.3	1.01	68.7	.08	226.7	96	3.3
3.4	70.7	.99	240.4	1.01	69.5	.09	236.3	.98	3.4
3.5	71.6	.08	250.5	1.01	70.4	.07	246.1	.97	3.5
3.6	72.4	08	260.6	1.02	71.1	.07	255.8	.99	3.6
3.7	73.2	.07	270.8	1.02	71.8	.07	265.7	.98	3.7
3.8	74.	.07	281.	1.04	72.5	.07	275.5	.99	3.8
3.9	74.7	.07	291.4	1.03	73.2	.07	285.4	1.01	3.9
4.	75.4		301.7		73.9		295.5		4.

TABLE 27.

Based on Kutter's formula, with $n = .035$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 5000=1.056 ft. per mile $s = .0002$				1 in 3333.3=1.584 ft. per mile $s = .0003$				$\sqrt{r}$ in feet
	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	
.4	18.3	.36	7.32	.37	18.8	.37	7.52	.38	.4
.5	21.9	.34	11.	.42	22.5	.34	11.3	.42	.5
.6	25.3	.30	15.2	.46	25.9	.30	15.5	.47	.6
.7	28.3	.30	19.8	.52	28.9	.29	20.2	.52	.7
.8	31.3	.26	25.	.55	31.8	.26	25.4	.56	.8
.9	33.9	.25	30.5	.59	34.4	.25	31.	.59	.9
1.	36.4	.24	36.4	.63	36.9	.23	36.9	.62	1.
1.1	38.8	.21	42.7	.64	39.2	.21	43.1	.65	1.1
1.2	40.9	.21	49.1	.68	41.3	.19	49.6	.67	1.2
1.3	43.	.19	55.9	.70	43.3	.19	56.3	.70	1.3
1.4	44.9	.18	62.9	.72	45.2	.17	63.3	71	1.4
1.5	46.7	.17	70.1	.73	46.9	.17	70.4	.74	1.5
1.6	48.4	.17	77.4	.78	48.6	.15	77.8	.74	1.6
1.7	50.1	.15	85.2	.77	50.1	.15	85.2	.77	1.7
1.8	51.6	.14	92.9	.78	51.6	.14	92.9	.78	1.8
1.9	53.	.14	100.7	.81	53.	.13	100.7	.79	1.9
2.	54.4	.13	108.8	.84	54.3	.12	108.6	.80	2.
2.1	55.7	.13	117.	.84	55.5	.12	116.6	.81	2.1
2.2	57.	.12	125.4	.85	56.7	.11	124.7	.82	2.2
2.3	58.2	.11	133.9	.84	57.8	.11	132.9	.85	2.3
2.4	59.3	.11	142.3	.87	58.9	.10	141.4	.84	2.4
2.5	60.4	.10	151.	.86	59.9	.10	149.8	.85	2.5
2.6	61.4	.10	159.6	.89	60.9	.09	158.3	.86	2.6
2.7	62.4	.09	168.5	.87	61.8	.09	166.9	.87	2.7
2.8	63.3	.09	177.2	.90	62.7	.09	175.6	.88	2.8
2.9	64.2	.09	186.2	.91	63.6	.08	184.4	.88	2.9
3.	65.1	.09	195.3	.93	64.4	.08	193.2	.89	3.
3.1	66.	08	204.6	.92	65.2	.07	202.1	.88	3.1
3.2	66.8	.07	213.8	.90	65.9	.08	210.9	.92	3.2
3.3	67.5	.08	222.8	.94	66.7	07	220.1	.91	3.3
3.4	68.3	.07	232.2	.93	67.4	.06	229.2	.89	3.4
3.5	69.	.07	241.5	.94	68.	.07	238.1	.91	3.5
3.6	69.7	.07	250.9	.94	68.7	.06	247.2	.92	3.6
3.7	70.4	.06	260.3	.95	69.3	.06	256.4	.92	3.7
3.8	71.	.06	269.8	.95	69.9	.06	265.6	.92	3.8
3.9	71.6	.06	279.3	.96	70.5	.05	274.8	.93	3.9
4.	72.2		288.9		71.		284.1		4.

TABLE 27.

Based on Kutter's formula, with $n = .035$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$	1 in 2500=2.112 ft. per mile				1 in 1666.7=3.168 ft. per mile				$\sqrt{r}$
	$s = .0004$				$s = .0006$				
in feet	c	diff. .01	$c\sqrt{r}$	diff. .01	c	diff. .01	$c\sqrt{r}$	diff. .01	in feet
.4	19.1	.37	7.6	.38	19.4	.37	7.76	.39	.4
.5	22.8	.34	11.4	.43	23 1	.34	11.6	.43	.5
.6	26.2	.30	15.7	.47	26.5	.31	15.9	.48	.6
.7	29.2	.29	20.4	.53	29.6	.28	20.7	52	.7
.8	32.1	.26	25.7	.55	32.4	.26	25.9	.56	.8
.9	34.7	.24	31.2	.59	35.	.24	31.5	.59	.9
1.	37.1	.23	37.1	.62	37.4	.22	37 4	.61	1.
1.1	39.4	.21	43.3	.65	39.6	.21	43.5	.65	1.1
1.2	41.5	.20	49.8	.68	41.7	.19	50.	.67	1.2
1.3	43.5	.18	56.6	68	43.6	18	56.7	.69	1.3
1.4	45.3	.17	63.4	.71	45.4	17	63.6	.71	1.4
1.5	47.	.16	70.5	.73	47.1	16	70.7	.72	1.5
1.6	48.6	.15	77.8	.74	48.7	.15	77.9	.74	1.6
1.7	50.1	.15	85.2	.77	50.2	.14	85.3	.76	1.7
1.8	51.6	.13	92.9	.76	51.6	.13	92.9	76	1.8
1.9	52.9	.13	100.5	.79	52.9	.13	100 5	.79	1.9
2.	54.2	.12	108.4	.79	54.2	.11	108.4	.77	2.
2.1	55.4	.12	116.3	.82	55.3	.12	116.1	.82	2.1
2.2	56.6	.11	124.5	.82	56.5	.10	124.3	.80	2.2
2.3	57.7	.10	132.7	.82	57.5	.10	132.3	.81	2.3
2.4	58.7	.10	140.9	.84	58.5	.10	140.4	.84	2.4
2.5	59.7	.10	149.3	.85	59.5	.09	148.8	.82	2.5
2.6	60.7	.09	157.8	.85	60.4	.09	157	.85	2.6
2.7	61.6	.08	166.3	.84	61.3	.08	165.5	.84	2.7
2.8	62.4	.08	174.7	.86	62.1	.08	173.9	.85	2.8
2.9	63.2	.08	183.3	.87	62.9	.08	182.4	.87	2.9
3.	64.	.08	192.	.89	63.7	.07	191 1	.85	3.
3.1	64.8	.07	200.9	.87	64.4	.07	199.6	.87	3.1
3.2	65.5	.07	209.6	.89	65.1	.06	208.3	.85	3.2
3.3	66.2	.07	218.5	.90	65.7	.07	216 8	.90	3.3
3.4	66.9	.06	227.5	.89	66.4	.06	225.8	.87	3.4
3.5	67.5	.06	236.4	.89	67.	.06	234.5	.89	3.5
3.6	68 1	.06	245.3	.90	67.6	.06	243.4	.88	3.6
3.7	68.7	.06	254.3	.91	68.2	.05	252.2	.90	3.7
3.8	69.3	.06	263.4	.91	68.7	.06	261 2	.89	3.8
3.9	69.9	.05	272.5	.91	69.3	.05	270 1	.90	3.9
4.	70.4		281.6		69.8		279 1		4.

TABLE 27.

Based on Kutter's formula, with $n = .035$. Values of the factors c and $c\sqrt{r}$ for use in the formulæ

$$v = c\sqrt{rs} = c \times \sqrt{r} \times \sqrt{s} = c\sqrt{r} \times \sqrt{s}$$

All slopes greater than 1 in 1000 have the same co-efficient as 1 in 1000.

$\sqrt{r}$ in feet	1 in 1250=4.224 ft. per mile, $s = .0008$: c	diff. .01	$c\sqrt{r}$	diff. .01	1 in 1000=5.28 ft. per mile, $s = .001$: c	diff. .01	$c\sqrt{r}$	diff. .01	$\sqrt{r}$ in feet
.4	19.6	.37	7.8	.39	19 7	.37	7.88	.39	.4
.5	23.3	.34	11.7	.43	23.4	.34	11.7	.44	.5
.6	26.7	.31	16.	.49	26.8	.31	16.1	.48	.6
.7	29.8	.28	20.9	.52	29.9	.28	20.9	.53	.7
.8	32.6	.26	26.1	.56	32.7	.26	26.2	.56	.8
.9	35.2	.24	31.7	.59	35.3	23	31.8	58	.9
1.	37.6	.22	37.6	.62	37.6	.22	37.6	.62	1.
1.1	39.8	.20	43.8	64	39.8	.21	43.8	.65	1.1
1.2	41.8	.19	50.2	.66	41.9	.19	50.3	.66	1.2
1.3	43.7	.18	56.8	.69	43.8	.18	56.9	.69	1.3
1.4	45.5	.17	63.7	.71	45.6	.16	63.8	.70	1.4
1.5	47.2	.15	70.8	.71	47.2	.16	70.8	.73	1.5
1.6	48.7	.15	77.9	.74	48.8	.14	78.1	.72	1.6
1.7	50.2	.14	85.3	.76	50.2	.14	85.3	.76	1.7
1.8	51 6	.13	92.9	.76	51.6	.13	92.9	.76	1.8
1.9	52.9	.12	100.5	.77	52.9	.12	100.5	.77	1.9
2.	54 1	.12	108.2	.79	54.1	.11	108.2	.77	2.
2.1	55.3	.11	116.1	.80	55.2	.11	115.9	.80	2.1
2.2	56.4	.10	124.1	79	56.3	.11	123.9	81	2.2
2.3	57.4	.10	132.	.82	57.4	.10	132.	.82	2.3
2.4	58.4	.10	140.2	.83	58.4	.09	140 2	.81	2.4
2.5	59.4	09	148.5	.83	59.3	.09	148.3	.82	2.5
2.6	60.3	.08	156.8	.82	60.2	08	156 5	.82	2.6
2.7	61.1	08	165.	.83	61.	.08	164.7	.83	2.7
2.8	61.9	08	173.3	.85	61 8	.08	173	.85	2.8
2.9	62.7	08	181.8	.87	62.6	.07	181.5	.84	2.9
3.	63.5	.07	190.5	85	63.3	.07	189 9	.85	3.
3.1	64.2	07	199.	.87	64.	.07	198 4	.86	3.1
3.2	64.9	.06	207.7	.85	64.7	.07	207.	.88	3.2
3.3	65.5	06	216.2	85	65.4	.06	215 8	.86	3.3
3.4	66.1	06	224.7	.89	66.	.06	224 4	.86	3.4
3 5	66.7	.06	233.6	.88	66 6	.06	233.	.88	3.5
3.6	67.3	.06	242.4	.88	67.2	05	241 8	.87	3.6
3.7	67 9	.05	251.2	.88	67.7	.05	250.5	.88	3.7
3.8	68 4	.05	260.0	.89	68.2	.06	259.3	.90	3.8
3.9	68.9	.05	268.9	.89	68.8	.04	268.3	.87	3.9
4.	69.4		277 8		69.2		277.		4.

TABLE 28.

Value of $c\sqrt{r}$ to be used *only* in the application of the second type of Bazin's formula for *open channels* with an even lining of cut stone, brickwork, or other material with surfaces of equal roughness, exposed to the flow of water. This formula is:—

$$v = c\sqrt{r} \times \sqrt{s}$$

$$\text{where } c = \sqrt{1 \div .000013\left(4.354 + \frac{1}{r}\right)}$$

Hydraulic mean depth in feet, r.	$c\sqrt{r}$	Hydraulic mean depth in feet, r.	$c\sqrt{r}$	Hydraulic mean depth in feet, r.	$c\sqrt{r}$	Hydraulic mean depth in feet, r.	$c\sqrt{r}$
.104	23.710	.396	65.756	1.062	122.82	2.250	187.77
.125	27.617	.417	68.159	1.125	127.05	2.375	193.36
.146	31.284	.437	70.337	1.187	131.	2.500	198.83
.167	34.569	.458	72.615	1.250	135.03	2.750	209.31
.187	38.147	.479	74.764	1.312	138.93	3.	219.36
.208	41.327	.500	76.907	1.375	142.79	3.250	228.98
.229	44.484	.562	83.048	1.437	146.42	3.500	238.18
250	47 430	.625	88.772	1.500	149.90	3.750	246.96
.271	50.267	.687	94.315	1.625	156.83	4.	255.58
292	53.077	.750	99.573	1.750	163.46	4.250	263.81
.312	55.783	.812	104.53	1.875	169.80	4.500	271.87
.333	58.346	.875	109.35	2.	175.99	4.750	279.78
.354	60.898	.937	114.	2.125	182.02	5.	287.30
.375	63.336	1.	118.50				

TABLE 29.

Giving the length of *two* side slopes of a trapezoidal channel. The side slopes plus the bed width are equal to the perimeter.

Depth in Feet	½ to 1	1 to 1	1½ to 1	2 to 1
.5	1.118	1.414	1.803	2.236
.75	1.677	2.121	2.704	3.354
1.	2.236	2.828	3.606	4.472
1.25	2.795	3.535	4.507	5.590
1.5	3.354	4.242	5.408	6.708
1.75	3.913	4.949	6.310	7.826
2.	4.472	5.656	7.212	8.944
2.25	5.031	6.345	8.112	10.062
2.5	5.590	7.070	9.014	11.181
2.75	6.149	7.778	9.916	12.299
3.	6.708	8.484	10.816	13.417
3.25	7.267	9.192	11.718	14.535
3.5	7.816	9.899	12.618	15.653
3.75	8.385	10.606	13.522	16.771
4.	8.944	11.312	14.422	17.889
4.25	9.503	12.021	15.324	19.006
4.5	10.062	12.728	16.226	20.124
4.75	10.621	13.435	17.126	21.242
5.	11.180	14.142	18.028	22.360
5.25	11.739	14.849	18.930	23.478
5.5	12.298	15.556	19.830	24.596
5.75	12.857	16.263	20.732	25.714
6.	13.416	16.970	21.634	26.833
6.25	13.975	17.678	22.536	27.951
6.5	14.534	18.385	23.436	29.069
6.75	15.093	19.092	24.338	30.187
7.	15.652	19.800	25.240	31.306
7.25	16.211	20.506	26.140	32.423
7.5	16.770	21.213	27.042	33.542
7.75	17.329	21.920	27.944	34.660
8.	17.888	22.627	28.844	35.778
8.25	18.447	23.334	29.746	36 896
8.5	19.006	24.042	30.648	38.014
8.75	19.565	24.749	31.550	39.132
9.	20.124	25.456	32.450	40 250
10.	22 360	28.284	36, 056	44.720
11.	24.596	31.112	39 662	49.194
12.	26.832	33.941	43.268	53.664
13.	29.068	36.769	46.872	58.139
14.	31.304	39.598	50.748	62.611
15.	33.540	42.626	54.080	67.083
16.	35.776	45.254	57.690	71.555

TABLE 30.

Giving velocities and discharges of trapezoidal channels in earth, according to Bazin's formula (37), for channels in earth:—

$$v = \sqrt{1 \div .00035\left(.2438 + \frac{1}{r}\right)} \times \sqrt{rs}$$

Side Slopes 1 to 1. v = **mean velocity in feet per second, and Q = discharge in cubic feet per second.**

(Professional Papers on Indian Engineering, Volume V, Second Series.)

Depth in feet.	Slope 1 in	Bed width 3 ft.		Bed width 4 ft.		Bed width 5 ft.		Bed width 6 ft.	
		v	Q	v	Q	v	Q	v	Q
1.	2500	.679	2.72	.721	3.61	.752	4.51	.776	5.43
1.	2857	.635	2.54	.675	3.37	.704	4.22	.726	5.08
1.	3333	.588	2.35	.625	3.12	.651	3.91	.672	4.70
1.	4000	.537	2.15	.570	2.85	.595	3.57	.613	4.29
1.	5000	.480	1.92	.510	2.55	.532	3.19	.549	3 84
1.	6666	.416	1.66	.442	2.20	.461	2.76	.475	3.33
1.5	2500	.899	6.07	.959	7 92	1.01	9.81	1.04	11.73
1.5	2857	.841	5.68	.897	7 40	.941	9.17	.976	10.97
1 5	3333	.779	5.26	.831	6.85	.871	8.49	.903	10.16
1.5	4000	.711	4.80	.758	6.26	.795	7.75	824	9.28
1.5	5000	.636	4.29	.678	5.60	.711	6.93	.737	8.30
1.5	6666	.511	3.72	.588	4.85	616	6.01	.639	7.18
2.	2500	1 09	10.91	1.16	13.96	1.22	17.11	1.27	20.32
2.	2857	1.02	10.20	1.09	13.06	1.14	16.01	1.19	19.01
2.	3333	.945	9.45	1.01	12.09	1.06	14.82	1.10	17.60
2.	4000	.862	8.62	.920	11.04	.966	13.53	1.	16.07
2.	5000	.771	7.71	.823	9 88	.864	12.10	.898	14.37
2.	6666	668	6.68	.713	8.55	.749	10.48	.778	12.44
2.5	2500	1.26	17.39	1.35	21.88	1.41	26.52	1.47	31.26
2.5	2857	1.18	16.27	1.26	20.47	1.32	24.80	1.38	29.23
2.5	3333	1.09	15.06	1 17	18.95	1.22	22.96	1.27	27 07
2.5	4000	1.	13.74	1.06	17.30	1.12	20.96	1.16	24 71
2.5	5000	.894	12.29	.952	15.47	1.	18.75	1.04	22.10
2.5	6666	.774	10.65	.825	13.40	.866	16.24	.901	19.14
3.	2500	1.43	25.65	1.51	31.79	1.59	38.13	1.65	44 60
3.	2857	1.35	23.99	1.42	29.74	1.49	35.67	1.55	41.72
3.	3333	1.23	22.21	1.31	27.54	1.38	33.02	1.43	38.64
3.	4000	1.13	20.27	1.20	25.14	1.26	30.15	1.31	35 26
3.	5000	1.01	18.13	1.07	22.49	1.12	26.97	1.17	31.54
3.	6666	.873	15.71	.927	19.47	973	23.35	1.01	27.32
3.5	2500	1.58	35.87	1.67	43.86	1.75	52.08	1.82	60.51
3.5	2857	1.47	33.50	1.56	41.02	1.64	48.72	1 70	56.60
3.5	3333	1.37	31 07	1.45	37.98	1.52	45.11	1.58	52.40
3.5	4000	1.25	28.36	1.32	34.70	1.38	41.18	1.44	47.83
3 5	5000	1.12	25.37	1.18	31.01	1.24	36 83	1.29	42.79
3.5	6666	.966	21.97	1.02	26.86	1.07	31.89	1.11	37.05

TABLE 30.

Giving velocities and discharges of trapezoidal channels in earth, according to Bazin's formula (37), for channels in earth:—

$$v = \sqrt{1 \div .00035\left(.2438 + \frac{1}{r}\right)} \times \sqrt{rs}$$

Side Slopes 1 to 1. v = mean velocity in feet per second, and Q = discharge in cubic feet per second.

Depth in feet.	Slope 1 in	Bed width 7 ft.		Bed width 8 ft.		Bed width 9 ft.		Bed width 10 ft.	
		v	Q	v	Q	v	Q	v	Q
1.	2500	.795	6.36	.810	7.29	.823	8.23	.834	9.17
1.	2857	.744	5.95	.758	6.82	.770	7.70	.780	8.58
1.	3333	.688	5.51	.702	6.32	.713	7.13	.722	7.94
1.	4000	.628	5.03	.641	5.77	.651	6.51	.659	7.25
1.	5000	.562	4.50	.573	5.16	.582	5.82	.590	6.48
1.	6666	.487	3.89	.496	4.47	.504	5.04	.511	5.62
1.5	2500	1.07	13.68	1.10	15.65	1.12	17 63	1.14	19.63
1.5	2857	1.	12.79	1.03	14.64	1.05	16.49	1.06	18.35
1.5	3333	.929	11.85	.942	13.55	.970	15.27	.985	17.
1.5	4000	.848	10.82	.869	12.37	885	13.93	.899	15.52
1.5	5000	.759	9.68	.777	10.08	.792	12.47	.804	13.88
1.5	6666	.657	8.38	.673	9.59	.686	10.80	.697	12.02
2.	2500	1.31	23.58	1.34	26.87	1 37	30.20	1.40	33.56
2.	2857	1.23	22.06	1.26	25 13	1.28	28.25	1.31	31.39
2.	3333	1.13	20.42	1.16	23.27	1.19	26 16	1.21	29.06
2.	4000	1.04	18.64	1.06	21.24	1.09	23.88	1.11	26.53
2.	5000	.926	16.68	.950	19.01	.971	21.36	.989	23.73
2.	6666	.802	14.44	.823	16.46	.841	18.50	.856	20.55
2.5	2500	1.52	36.07	1.56	40.95	1.60	45.88	1.63	50.85
2.5	2857	1.42	33.74	1.46	38.30	1.49	42.91	1.52	47.57
2.5	3333	1.32	31.24	1.35	35.46	1.38	39.73	1.41	44.04
2.5	4000	1.20	28.51	1.23	32.37	1.26	36.27	1.29	40.20
2.5	5000	1.07	25.51	1.10	28.96	1.13	32.44	1.15	35.96
2.5	6666	.930	22.09	.955	25.08	.977	28.10	.997	31.14
3.	2500	1.71	51.21	1.76	57.90	1.80	64.66	1.83	71.48
3.	2857	1.60	47.90	1.64	54.15	1.68	60.48	1.71	66.86
3.	3333	1.48	44.35	1.52	50.14	1.56	55.99	1.59	61.90
3.	4000	1.35	40.48	1.39	45.77	1.42	51.12	1.45	56.51
3.	5000	1.21	36.21	1.24	40.94	1.27	45.72	1.30	50.54
3.	6666	1 05	31.36	1.07	35.45	1.10	39.59	1.12	43.77
3.5	2500	1.88	69.07	1.93	77.77	1.98	86.57	2.02	95.46
3.5	2857	1.76	64.61	1.81	72.74	1.85	80.97	1.89	89.29
3.5	3333	1.63	59.82	1.67	67.35	1.71	74.79	1.75	82.67
3.5	4000	1.49	54.60	1.53	61.48	1.56	68.44	1.60	75.46
3.5	5000	1.33	48.84	1.37	54.99	1.40	61.21	1.43	67.50
3.5	6666	1.15	42.30	1.18	47.62	1.21	53.01	1.24	58.45

TABLE 30.

Giving velocities and discharges of trapezoidal channels in earth, according to Bazin's formula (37), for channels in earth:—

$$v = \sqrt{1 \div .00035\left(.2438 + \frac{1}{r}\right)} \times \sqrt{rs}$$

Sides Slopes 1 to 1. v = mean velocity in feet per second, and Q = discharge in cubic feet per second.

Depth in feet.	Slope 1 in	Bed width 11 ft.		Bed width 12 ft.		Bed width 13 ft.		Bed width 14 ft.	
		v	Q	v	Q	v	Q	v	Q
1.	2500	.843	10.11	.850	11.06	.858	12.01	.864	12.95
1.	2857	.788	9.46	.796	10.34	.802	11.23	.808	12.12
1.	3333	.730	8.76	.737	9.58	.743	10.40	.748	11.22
1.	4000	.666	8.	.673	8.74	.678	9.49	.683	10.24
1.	5000	.596	7.15	.602	7.82	.606	8.49	.611	9.16
1.	6666	.516	6.19	.521	6.77	.525	7.35	.529	7.93
1.5	2500	1.15	21 63	1.17	23.64	1.18	25.65	1.19	27.67
1.5	2857	1.08	20.23	1.10	22.11	1.10	23.99	1.11	25.88
1.5	3333	.999	18.73	1.01	20.47	1.02	22.21	1.03	23.96
1.5	4000	.912	17.10	.923	18.69	.932	20.28	.941	21.87
1.5	5000	.816	15.29	.825	16.71	.834	18.14	842	19.57
1.5	6666	.706	13.24	.715	14.47	.722	15.71	.729	16.94
2.	2500	1.42	36.92	1.44	40.31	1.46	43.71	1.47	47.12
2.	2857	1.33	34.54	1.35	37.70	1.36	40.88	1.38	44.07
2	3333	1.23	31.98	1.25	34.91	1.26	37.85	1.28	40.81
2.	4000	1.12	29.19	1.14	31.87	1.15	34.55	1.16	37.25
2.	5000	1.	26.11	1.02	28.50	1.03	30 91	1.04	33.32
2.	6666	.870	22.61	.882	24.68	.892	26.76	.902	28.85
2.5	2500	1.65	55.84	1.68	60.89	1.70	65.95	1.72	71.03
2.5	2857	1.55	52.24	1.57	56.96	1.59	61 69	1.61	66 44
2.5	3333	1.43	48.36	1.45	52.73	1.47	57.11	1.49	61.51
2.5	4000	1.31	44.15	1.33	48.14	1.35	52.13	1.36	56.15
2.5	5000	1.17	39.49	1.19	43.06	1.20	46 63	1.22	50.23
2.5	6666	1.01	34.20	1.03	37.29	1.04	40.39	1.05	43.49
3.	2500	1.87	78.36	1.90	85.28	1.92	92.24	1.95	99.23
3.	2857	1.75	73.29	1.77	79.77	1.80	86.28	1.82	92.82
3.	3333	1.62	67.86	1.64	73.85	1.66	79.88	1.69	85.94
3.	4000	1.48	61.95	1.50	67.42	1.52	72.92	1.54	78.45
3.	5000	1.32	55.41	1.34	60.30	1.36	65.23	1.38	70.17
3.	6666	1.14	47.99	1.16	52.22	1.18	56.49	1.19	60.77
3.5	2500	2.06	104.42	2.09	113.44	2.12	122.53	2.15	131.66
3.5	2357	1.92	97.67	1.96	106.11	1.98	114.61	2.01	123.15
3.5	3333	1.78	90.43	1.81	98.24	1.84	106.11	1.86	114.01
3.5	4000	1.63	82.55	1.65	89.68	1.68	96.86	1.70	104.08
3.5	5000	1.45	73.84	1.48	80 22	1.50	86.64	1.52	93.10
3.5	6666	1.26	63.94	1.28	69.47	1.30	75.03	1.32	80.62

TABLE 30.

Giving velocities and discharges of trapezoidal channels in earth, according to Bazin's formula (37), for channels in earth:—

$$v = \sqrt{1 \div .00035\left(.2438 + \frac{1}{r}\right)} \times \sqrt{rs}$$

Side Slopes 1 to 1. v = mean velocity in cubic feet per second, and Q = discharge in cubic feet per second.

Depth in feet.	Slope 1 in	Bed width 15 ft.		Bed width 16 ft.		Bed width 18 ft.		Bed width 20 ft.	
		v	Q	v	Q	v	Q	v	Q
1.5	2500	1.20	29.69	1.21	31.72	1.22	35.78	1.24	39.85
1.5	2857	1.12	27.77	1.13	29.67	1.14	33.47	1.16	37.28
1.5	3333	1.04	25.71	1.05	27.47	1.06	30.99	1.07	34.52
1.5	4000	.948	23.47	.955	25.08	.967	28.29	.977	31.51
1.5	5000	.848	21.	.854	22.43	.865	25.30	.874	28.18
1.5	6666	.735	18.18	.740	19.42	.749	21.91	.757	24.41
2.	2500	1.49	50.54	1.50	53.96	1.52	60.84	1.54	67.74
2.	2857	1.39	47.27	1.40	50.48	1.42	56.91	1.44	63.36
2.	3333	1.29	43.77	1.30	46.74	1.32	52.69	1.33	58.66
2.	4000	1.18	39.95	1.19	42.66	1.20	48.09	1.22	53.55
2.	5000	1.05	35.74	1.06	38.16	1.08	43.02	1.09	47.90
2.	6666	.910	30.95	.918	33.04	.931	37.26	.943	41.48
2.5	2500	1.74	76.13	1.76	81.24	1.79	91.50	1.81	101.80
2.5	2857	1.63	71.21	1.64	75.99	1.67	85.58	1.69	95.22
2.5	3333	1.51	65.93	1.50	70.36	1.55	79.24	1.57	88.16
2.5	4000	1.38	60.18	1.39	64.23	1.41	72.33	1.43	80.48
2.5	5000	1.23	53.83	1.24	57.45	1.26	64.70	1.28	71.98
2.5	6666	1.07	46.62	1.08	49.75	1.09	56.03	1.11	62.34
3.	2500	1.97	106.26	1.99	113.30	2.02	127.46	2.05	141.68
3.	2857	1.84	99.40	1.86	105.98	1.89	119.22	1.92	132.53
3.	3333	1.70	92.02	1.72	98.12	1.75	110.38	1.78	122.70
3.	4000	1.56	84.	1.57	89.57	1.60	100.76	1.62	112.01
3.	5000	1.39	75.14	1.41	80.12	1.43	90.13	1.45	100.19
3.	6666	1.21	65.07	1.22	69.38	1.24	78.05	1.26	86.76
3.5	2500	2.17	140.82	2.20	150.03	2.24	168.54	2.28	187.15
3.5	2857	2.03	131.72	2.06	140.34	2.09	157.64	2.13	175.05
3.5	3333	1.88	121.95	1.90	129.93	1.94	145.95	1.97	162.07
3.5	4000	1.72	111.33	1.74	118.61	1.77	133.24	1.80	147.95
3.5	5000	1.54	99.58	1.55	106.09	1.58	119.17	1.61	132.33
3.5	6666	1.33	86.23	1.35	91.87	1.37	103.21	1.39	114.60
4.	2500	2.37	179.77	2.39	191.34	2.44	214.61	2.48	238.03
4.	2857	2.21	168.15	2.24	178.97	2.28	200.74	2.32	222.64
4.	3333	2.05	155.68	2.07	165.70	2.11	185.86	2.15	206.13
4.	4000	1.87	142.12	1.89	151.26	1.93	169.66	1.96	188.17
4.	5000	1.67	127.12	1.69	135.30	1.72	151 76	1.75	168.31
4.	6666	1.45	110.08	1.46	117.17	1.49	131.42	1.52	145.76

TABLE 31.

Velocities and discharges in trapezoidal channels based on Kutter's formula with $n = .025$. Side slopes 1 horizontal to 1 vertical.

Bed Width 30 Feet.				Bed Width 40 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
2	1500	2.203	141.	2	1500	2.242	188.3
2	2000	1.905	121.9	2	2000	1.941	163.
2	3000	1.539	98.5	2	3000	1.574	132.2
2	5000	1.231	78.8	2	5000	1.215	102.1
3	1500	2.856	282.7	3	1500	2.923	377.
3	2000	2.471	244.6	3	2000	2.535	327.
3	3000	2.013	199.3	3	3000	2.062	266.
3	5000	1.556	154.	3	5000	1.596	205.9
4	1500	3.396	461.8	4	1500	3.497	615.4
4	2000	2.936	399.3	4	2000	2.982	524.8
4	3000	2.401	326.6	4	3000	2.473	435.2
4	5000	1.858	252.7	4	5000	1.889	332.5
5	1500	3.859	675.3	5	1500	4.112	925.2
5	2000	3.334	585.2	5	2000	3.454	777.1
5	3000	2.736	478.8	5	3000	2.826	635.8
5	5000	2.123	371.5	5	5000	2.194	493.6
Bed Width 50 Feet.				Bed Width 60 Feet.			
2	1500	2.268	235.8	2	1500	2.294	284.4
2	2000	1.965	204.4	2	2000	1.979	245.4
2	3000	1.765	183.5	2	3000	1.611	199.7
2	5000	1.229	127.8	2	5000	1.238	153.5
3	1500	2.968	472.	3	1500	3.	567.
3	2000	2.570	408.6	3	2000	2.600	491.4
3	3000	2.096	333.2	3	3000	2.127	402.
3	5000	1.618	257.3	3	5000	1.638	309.6
4	1500	3.559	768.7	4	1500	3.607	923.4
4	2000	3.085	666.3	4	2000	3.123	799.5
4	3000	2.537	548.	4	3000	2.553	653.5
4	5000	1.953	421.8	4	5000	1.980	506.9
5	1500	4.068	1118.7	5	1500	4.136	1344.2
5	2000	3.528	970.2	5	2000	3.582	1164.1
5	3000	2.887	793.9	5	3000	2.935	953.8
5	5000	2.243	616.8	5	5000	2.277	740.

TABLE 31.

Velocities and discharges in trapezoidal channels based on Kutter's formula with $n = 0.25$. Side slopes 1 horizontal to 1 vertical.

Depth in feet.	Slope 1 in	Velocity in feet per second	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
Bed Width 70 Feet.				**Bed Width 80 Feet.**			
3	2000	2.622	574.2	3	2000	2.637	656.6
3	3500	1.976	432.7	3	3500	1.989	495.2
3	7500	1.344	294.3	3	7500	1.353	336.9
3	10000	1.163	254.7	3	10000	1.169	291.1
4	2000	3.152	933.	4	2000	3.175	1066.8
4	3500	2.387	706.5	4	3500	2.404	807.7
4	7500	1.635	483.9	4	7500	1.648	553.7
4	10000	1.418	419.7	4	10000	1.429	480.1
5	2000	3.621	1357.9	5	2000	3.653	1552.5
5	3500	2.746	1029.7	5	3500	2.77	1177.2
5	7500	1.89	708.7	5	7500	1.909	811.3
5	10000	1.643	616.1	5	10000	1.657	704.2
6	2000	4.040	1842.2	6	2000	4.080	2105.3
6	3500	3.066	1398.1	6	3500	3.099	1599.
6	7500	2.121	967.1	6	7500	2.144	1106.3
6	10000	1.848	842.7	6	10000	1.869	964.4
Bed Width 90 Feet.				**Bed Width 100 Feet.**			
3	2000	2.649	739.1	3	2000	2.657	821.
3	3500	1.998	557.4	3	3500	2.004	619.2
3	7500	1.359	379.1	3	7500	1.364	421.4
3	10000	1.175	327.8	3	10000	1.180	364.6
4	2000	3.196	1201.7	4	2000	3.208	1334.5
4	3500	2.419	909.5	4	3500	2.431	1011.3
4	7500	1.658	623.4	4	7500	1.667	693.4
4	10000	1.439	541.1	4	10000	1.446	601.5
5	2000	3.677	1746.6	5	2000	3.702	1943.5
5	3500	2.79	1325.2	5	3500	2.806	1473.1
5	7500	1.923	913.4	5	7500	1.935	1015.8
5	10000	1.670	793.2	5	10000	1.682	883.
6	2000	4.120	2373.1	6	2000	4.140	2633.
6	3500	3.122	1798.2	6	3500	3.143	1998.9
6	7500	2.161	1244.7	6	7500	2.176	1383.9
6	10000	1.888	1087.5	6	10000	1.898	1199.8

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Sides slopes ½ horizontal to 1 vertical.

Bed Width 1 Foot.				Bed Width 2 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second	Discharge in cubic feet per second.
1.5	266	1	.625	.5	380	1	1.125
1.5	66	2	1.25	.5	95	2	2.25
1.5	30	3	1.875	.5	42	3	3.375
1.5	17	4	2.5	.5	24	4	4.5
1.	542	1	1.5	1.	870	1	2 5
1.	135	2	3.	1.	217	2	5.
1.	60	3	4.5	1.	97	3	7·5
1.	34	4	6.	1.	54	4	10.
1.5	911	1	2.625	1.5	1340	1	4.125
1.5	228	2	5.25	1.5	335	2	8.25
1.5	101	3	7.875	1.5	149	3	12·375
1.5	57	4	10.5	1.5	84	4	16.5
				2.	1752	1	6.
				2.	438	2	12.
				2.	194	3	18.
				2.	110	4	24.
Bed Width 3 Feet.				Bed Width 4 Feet.			
.5	448	1	1.625	1.	1195	1	4.5
.5	112	2	3.25	1.	300	2	9.
.5	50	3	4.875	1.	133	3	13.5
.5	28	4	6.5	1.	75	4	18.
1.	1070	1	3.5	1.25	1536	1	5.8
1.	268	2	7.	1.25	387	2	11.6
1.	119	3	10.5	1.25	172	3	17.3
1.	67	4	14.	1.25	97	4	23.1
1.5	1657	1	5.625	1.5	1859	1	7 1
1.5	414	2	11.25	1.5	473	2	14.2
1.5	184	3	16.875	1.5	210	3	21.4
1.5	104	4	22.5	1.5	118	4	28.5
2.	2216	1	8.	2.	2570	1	10.
2.	554	2	16.	2.	660	2	20.
2.	246	3	24.	2.	293	3	30.
2.	138	4	32.	2.	165	4	40.
2.5	2790	1	10.62	2.5	3188	1	13.1
2.5	698	2	21.25	2.5	822	2	26.3
2 5	310	3	31.88	2.5	365	3	39.4
2.5	174	4	42.5	2.5	206	4	52.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes $\frac{1}{2}$ horizontal to 1 vertical.

Bed Width 6 Feet.				Bed Width 8 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
1.	1380	1	6.5	1.	1459	1	8.5
1.	348	2	13.	1.	373	2	17.
1.	155	3	19.5	1.	166	3	25.5
1.	87	4	26.	1.	93	4	34.
1.25	1798	1	8.3	1.25	1984	1	10.8
1.25	457	2	16.6	1.25	504	2	21.6
1.25	203	3	24.8	1.25	224	3	32.3
1.25	114	4	33.1	1.25	126	4	43.1
1.5	2230	1	10.1	1.5	2433	1	13.1
1.5	570	2	20.2	1.5	624	2	26.3
1.5	253	3	30.4	1.5	277	3	39.4
1.5	142	4	40.5	1.5	156	4	52.5
1.75	2671	1	12.	1.75	2947	1	15.5
1.75	680	2	24.	1.75	758	2	31.
1.75	302	3	36.1	1.75	337	3	46.5
1.75	170	4	48.1	1.75	190	4	62.1
2.	3101	1	14.	2.	3451	1	18.
2.	800	2	28.	2.	889	2	36.
2.	356	3	42.	2.	395	3	54.
2.	200	4	56.	2.	222	4	72.
2.25	3533	1	16.	2.25	3886	1	20.5
2 25	912	2	32.	2.25	1006	2	41.
2.25	405	3	48.	2.25	447	3	61.6
2.25	228	4	64.1	2.25	252	4	82.1
2.5	3895	1	18.1	2.5	4385	1	23.1
2.5	1006	2	36.2	2.5	1134	2	46.2
2.5	447	3	54.4	2.5	504	3	69.4
2.5	252	4	72.5	2.5	283	4	92.5
2.75	4292	1	20.3	2.75	4906	1	25.8
2.75	1107	2	40.6	2.75	1266	2	51.6
2.75	492	3	60.8	2.75	563	3	77.3
2.75	277	4	80.1	2.75	317	4	103.1
3.	4672	1	22.5	3.	5348	1	28.5
3.	1213	2	45.	3.	1382	2	57.
3.	539	3	67.5	3.	615	3	85.5
3.	303	4	90.	3.	346	4	114.

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes ½ horizontal to 1 vertical.

Bed Width 10 Feet.				Bed Width 12 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
1.5	2651	1	16.1	1.5	2803	1	19.1
1.5	680	2	32.3	1.5	718	2	38.3
1.5	302	3	48.4	1.5	319	3	57.4
1.5	170	4	64.5	1.5	180	4	76.5
1.75	3190	1	19.	1.75	3368	1	22.5
1.75	822	2	58.	1.75	866	2	45.1
1.75	365	3	57.1	1.75	385	3	67.6
1.75	206	4	76.1	1.75	217	4	90.1
2.	3731	1	22.	2.	3953	1	26.
2.	958	2	44.	2.	1030	2	52.
2.	426	3	66.	2.	458	3	78.
2.	239	4	88.	2.	258	4	104.
2.25	4275	1	25.	2.25	4586	1	29.5
2.25	1107	2	50.	2.25	1186	2	59.1
2.25	492	3	75.1	2.25	528	3	88.6
2.25	277	4	100.1	2.25	297	4	118.1
2.5	4826	1	28.1	2.5	5128	1	33.1
2.5	1237	2	56.3	2.5	1323	2	66.2
2.5	551	3	84.4	2.5	588	3	99.4
2.5	310	4	112.5	2.5	331	4	132.5
2.75	5352	1	31.3	2.75	5728	1	36.8
2.75	1383	2	62.6	2.75	1467	2	73.6
2.75	615	3	93.8	2.75	655	3	110.3
2.75	346	4	125.1	2.75	368	4	147.1
3.	5945	1	34.5	3.	6328	1	40.5
3.	1528	2	69.	3.	1625	2	81.
3.	682	3	103.5	3.	725	3	121.5
3.	384	4	138.	3.	408	4	162.
3.25	6503	1	37.8	3.25	7023	1	44.3
2.25	1658	2	75.6	3.25	1794	2	88.6
3.25	740	3	113.3	3.25	800	3	132.8
3.25	416	4	151.1	3.25	450	4	177.1
3.5	6992	1	41.1	3.5	7577	1	48.1
3.5	1793	2	82.2	3.5	1930	2	96.2
3.5	800	3	123.4	3.5	864	3	144 4
3.5	450	4	164.5	3.5	486	4	192.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes ½ horizontal to 1 vertical.

Bed Width 14 Feet.				Bed Width 16 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
1.5	2859	1	21.1	1.5	2948	1	25.1
1.5	738	2	44.2	1.5	758	2	50.2
1.5	328	3	66.3	1.5	337	3	75.3
1.5	185	4	88.5	1.5	189	4	100.5
1.75	3472	1	26.	1.75	3623	1	29.5
1.75	889	2	52.	1.75	935	2	59.
1.75	395	3	78.	1.75	415	3	88.5
1.75	222	4	104.	1.75	234	4	118.1
2.	4120	1	30.	2.	4293	1	34.
2.	1060	2	60.	2.	1110	2	68.
2.	470	3	90.	2.	492	3	102.
2.	264	4	120.	2.	277	4	136.
2.25	4678	1	34.	2.25	4898	1	38.5
2.25	1210	2	68.	2.25	1266	2	77.
2.25	539	3	102.	2.25	563	3	115.5
2.25	303	4	136.1	2.25	317	4	154.1
2.5	5364	1	38.1	2.5	5552	1	43.1
2.5	1383	2	76.2	2.5	1433	2	86.2
2.5	615	3	114.3	2.5	637	3	129.3
2.5	346	4	152.5	2.5	359	4	172.5
2.75	6084	1	42.3	2.75	6325	1	47.8
2.75	1559	2	84.6	2.75	1622	2	95.6
2.75	696	3	126.8	2.75	726	3	143.3
2.75	392	4	169.1	2.75	408	4	191.1
3.	6732	1	46.5	3.	7023	1	52.5
3.	1723	2	93.	3.	1794	2	105.
3.	770	3	139.5	3.	800	3	157.5
3.	433	4	186.	3.	450	4	210.
3.25	7427	1	50.8	3.25	7730	1	57.3
3.25	1896	2	101.6	3.25	1964	2	114.6
3.25	848	3	152.3	3.25	880	3	171.8
3.25	477	4	203.1	3.25	495	4	229.1
3.5	8013	1	55.1	3.5	8331	1	62.1
3.5	2045	2	110.2	3.5	2120	2	124.2
3.5	914	3	165.3	3.5	949	3	186.3
3.5	514	4	220.5	3.5	534	4	248.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes $\frac{1}{2}$ horizontal to 1 vertical.

Bed Width 18 Feet.				Bed Width 20 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
1.5	3124	1	28.1	1.5	3022	1	31.1
1.5	779	2	56.2	1.5	779	2	62.3
1.5	348	3	84.4	1.5	346	3	93.3
1.5	195	4	112.5	1.5	195	4	124.5
1.75	3713	1	33.	1.75	3713	1	36.5
1.75	958	2	66.	1.75	958	2	73.
1.75	426	3	99.1	1.75	426	3	109.6
1.75	240	4	132.1	1.75	240	4	146.1
2.	4385	1	38.	2.	4492	1	42.
2.	1130	2	76.	2.	1157	2	84.
2.	504	3	114.	2.	515	3	126.
2.	284	4	152.	2.	290	4	168.
2.25	5114	1	43.	2.25	5245	1	47.5
2.25	1320	2	86.	2.25	1352	2	95.
2.25	589	3	129.1	2.25	602	3	142.6
2.25	331	4	172.1	2.25	338	4	190.
2.5	5825	1	48.1	2.5	5935	1	53.1
2.5	1500	2	96.2	2.5	1528	2	106.2
2.5	668	3	144.4	2.5	682	3	159.3
2.5	376	4	192.5	2.5	384	4	212.5
2.75	6585	1	53.3	2.75	6737	1	58.8
2.75	1692	2	106.6	2.75	1726	2	117.6
2.75	755	3	159.8	2.75	770	3	176.3
2.75	425	4	213.1	2.75	433	4	235.1
3.	7285	1	58.5	3.	7427	1	64.5
3.	1862	2	117.	3.	1897	2	129.
3.	832	3	175.5	3.	848	3	193.
3.	468	4	234.	3.	477	4	258.
3.25	8028	1	63.8	3.25	8163	1	70.3
3.25	2056	2	127.6	3.25	2083	2	140.6
3.25	914	3	191.3	3.25	931	3	210.8
3.25	514	4	255.1	3.25	524	4	281.1
3.5	8807	1	69.1	3.5	8966	1	76.1
3.5	2251	2	138.2	3.5	2282	2	152.2
3.5	1000	3	207.4	3.5	1018	3	228.3
3.5	563	4	276.5	3.5	573	4	304.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes $\frac{1}{4}$ horizontal to 1 vertical.

Bed Width 25 Feet.				Bed Width 30 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
2.	4697	1	52.	2.	4797	1	62.
2.	1212	2	104.	2.	1237	2	124.
2.	541	3	156.	2.	551	3	186.
2.	304	4	208.	2.	310	4	248.
2.25	5489	1	58.8	2.25	5589	1	70.
2.25	1408	2	117.6	2.25	1435	2	140.
2.25	628	3	176.3	2.25	641	3	210.
2 25	353	4	235.1	2.25	361	4	280.
2.5	6197	1	65.6	2.5	6448	1	78.1
2.5	1586	2	131.2	2.5	1657	2	156.2
2.5	711	3	196.8	2.5	740	3	234 3
2.5	400	4	262.5	2.5	416	4	312.5
2.75	6992	1	72.5	2.75	7310	1	86.3
2.75	1792	2	145.	2.75	1866	2	172.6
2.75	800	3	217.6	2.75	832	3	258.8
2.75	450	4	290.1	2.75	468	4	345.1
3.	7878	1	79.5	3.	8168	1	94.5
3.	2008	2	159.	3.	2084	2	189.
3.	897	3	238.5	3.	931	3	283.5
3.	504	4	318.	3.	523	4	378.
3.5	9651	1	93.6	3.5	10007	1	111.1
3.5	2450	2	187.2	3.5	2531	2	222.2
3.5	1091	3	280.9	3.5	1127	3	333.3
3.5	614	4	374.5	3.5	634	4	444.5
4.	11308	1	108.	4.	11952	1	128.
4.	2840	2	216.	4.	2958	2	256.
4.	1263	3	324.	4.	1323	3	384.
4.	708	4	432.	4.	745	4	512.
4.5	13185	1	122.6	4.5	13831	1	145.1
4.5	3285	2	245.2	4.5	3436	2	290.2
4.5	1454	3	367.9	4.5	1522	3	435.3
4.5	818	4	490.5	4.5	856	4	580.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes $\frac{1}{2}$ horizontal to 1 vertical.

BED WIDTH 35 FEET.				BED WIDTH 40 FEET.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
2.	4886	1	72.	2.	5012	1	82.
2.	1266	2	144.	2.	1294	2	164.
2.	563	3	216.	2.	576	3	246.
2.	317	4	288.	2.	324	4	328.
2.25	5706	1	81.3	2.25	5853	1	92.5
2.25	1465	2	162.6	2.25	1504	2	185.
2.25	655	3	243.8	2.25	668	3	277.6
2.25	368	4	325.1	2.25	376	4	370.1
2.5	6601	1	90.6	2.5	6732	1	103.1
2.5	1691	2	181.2	2.5	1725	2	206.3
2.5	754	3	271.9	2.5	770	3	309.4
2.5	425	4	362.5	2.5	433	4	412.5
2.75	7261	1	100.	2.75	7725	1	113.8
2.75	1935	2	200.	2.75	1969	2	227.6
2.75	864	3	300.	2.75	880	3	341.3
2.75	486	4	400.	2.75	495	4	455.1
3.	8479	1	109.5	3.	8642	1	124.5
3.	2158	2	219.	3.	2199	2	249.
3.	965	3	328.5	3.	982	3	373.5
3.	543	4	438.	3.	552	4	498.
3.5	10381	1	128.6	3.5	10751	1	146.1
3.5	2630	2	257.2	3.5	2705	2	292.2
3.5	1164	3	385.8	3.5	1203	3	438.3
3.5	654	4	514.4	3.5	677	4	584.5
4.	12515	1	148.	4.	12776	1	168.
4.	3125	2	296.	4.	3163	2	336.
4.	1380	3	444.	4.	1406	3	504.
4.	782	4	592.	4.	791	4	672.
4.5	14505	1	167.6	4.5	14997	1	190.1
4.5	3591	2	335.2	4.5	3701	2	380,3
4.5	1591	3	502.9	4.5	1640	3	570.4
4.5	895	4	670.5	4.5	922	4	760.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes ½ horizontal to 1 vertical.

Bed Width 45 Feet.				Bed Width 50 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
2.	5013	1	92.	2.	5128	1	102.
2.	1294	2	184.	2.	1322	2	204.
2.	576	3	276.	2.	589	3	306.
2.	324	4	368.	2.	331	4	408.
2.25	5951	1	103.8	2.25	6086	1	115.
2.25	1527	2	207.6	2.25	1557	2	230.
2.25	682	3	311.3	2.25	697	3	345.
2.25	384	4	415.1	2.25	392	4	460.
2.5	6864	1	115.6	2.5	6999	1	128.1
2.5	1759	2	231.3	2.5	1794	2	256.3
2.5	785	3	346.9	2.5	800	3	384.4
2.5	442	4	462.5	2.5	450	4	512.5
2.75	7886	1	127.5	2.75	8039	1	141.3
2.75	2012	2	255.	2.75	2034	2	282.6
2.75	897	3	382.6	2.75	914	3	423.9
2.75	504	4	510.1	2.75	514	4	565.1
3.	8800	1	139.5	3.	8969	1	154.5
3.	2239	2	279.	3.	2275	2	309.
3.	998	3	418.5	3.	1018	3	463.5
3.	562	4	558.	3.	573	4	618.
3.5	10930	1	163.6	3.5	14130	1	181.1
3.5	2751	2	327.3	3.5	2796	2	362.2
3.5	1223	3	490.9	3.5	1243	3	543.4
3.5	688	4	654.5	3.5	699	4	724.5
4.	13180	1	188.	4.	13410	1	208.
4.	3272	2	376.	4.	3331	2	416.
4.	1454	3	564.	4.	1477	3	624.
4.	821	4	752.	4.	830	4	832.
4.5	15230	1	212.6	4.5	15707	1	235.1
4.5	3751	2	425.3	4.5	3866	2	470.2
4.5	1661	3	637.9	4.5	1707	3	705.4
4.5	935	4	850.5	4.5	960	4	940.5

TABLE 32.

Velocities and discharges in trapezoidal channels based on Kutter's formula, with $n = .03$. Side slopes ½ horizontal to 1 vertical.

Bed Width 60 Feet.				Bed Width 70 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
3.	2317	2	369.	3.	2356	2	429.
3.	1035	3	553.5	3.	1050	3	643.5
3.	583	4	738.	3.	593	4	858.
3.	373	5	922.5	3.	378	5	1072.5
3.25	2623	2	400.6	3.25	2661	2	465.6
3.25	1163	3	600.8	3.25	1183	3	698.4
3.25	654	4	801.1	3.25	665	4	931.1
3.25	419	5	1001.4	3.25	426	5	1163.9
3.5	2893	2	432.3	3.5	2949	2	502.3
3.5	1286	3	648.4	3.5	1305	3	753.4
3.5	723	4	864.5	3.5	734	4	1004.5
3.5	464	5	1080.6	3.5	470	5	1255.6
4.	3435	2	496.	4.	3488	2	576.
4.	1522	3	744.	4.	1544	3	864.
4.	856	4	992.	4.	869	4	1152.
4.	548	5	1240.	4.	556	5	1440.
4.5	3988	2	560.3	4.5	4094	2	650.3
4.5	1759	3	840.4	4.5	1807	3	975.4
4.5	989	4	1120.5	4.5	1017	4	1300.5
4.5	633	5	1400.6	4.5	651	5	1625.6
5.	4602	2	625.	5.	4653	2	725.
5.	2020	3	937.5	5.	2045	3	1087.5
5.	1133	4	1250.	5.	1148	4	1450.
5.	723	5	1562.5	5.	734	5	1812.5
6.	5785	2	756.	6.	5963	2	876.
6.	2538	3	1134.	6.	2584	3	1314.
6.	1406	4	1512.	6.	1440	4	1752.
6.	900	5	1890.	6.	922	5	2190.

TABLE 32.

Velocities and discharges in trapezoidal channels, based on Kutter's formula, with $n = .03$. Side slopes ½ horizontal to 1 vertical.

Bed Width 80 Feet.				Bed Width 90 Feet.			
Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.	Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
3.	2404	2	489.	3.	2403	2	549.
3.	1070	3	733.5	3.	1074	3	823.5
3.	603	4	978.	3.	603	4	1098.
3.	386	5	1222.5	3.	386	5	1372.5
3.25	2661	2	530.6	3.25	2704	2	595.6
3.25	1183	3	795.8	3.25	1203	3	893.3
3.25	665	4	1061.1	3.25	677	4	1191.1
3.25	426	5	1326.4	3.25	433	5	1488.9
3.5	2946	2	572.3	3.5	2982	2	642.3
3.5	1305	3	858.4	3.5	1326	3	963.4
3.5	734	4	1144.5	3.5	746	4	1284.5
3.5	470	5	1430.6	3.5	477	5	1605.6
4.	3541	2	656.	4.	3596	2	736.
4.	1567	3	984.	4.	1590	3	1104.
4.	882	4	1312.	4.	895	4	1472.
4.	564	5	1640.	4.	573	5	1840.
4.5	4167	2	740.3	4.5	4221	2	830.3
4.5	1835	3	1110.4	4.5	1859	3	1245.4
4.5	1030	4	1480.5	4.5	1045	4	1660.5
4.5	660	5	1850.6	4.5	668	5	2075.6
5.	4792	2	825.	5.	4833	2	925.
5.	2104	3	1237.5	5.	2139	3	1387.5
5.	1178	4	1650.	5.	1194	4	1850.
5.	754	5	2062.5	5.	764	5	2312.5
6.	6079	2	996.	6.	6175	2	1116.
6.	2649	3	1494.	6.	2682	3	1674.
6.	1477	4	1992.	6.	1488	4	2232.
6.	943	5	2490.	6.	952	5	2790.

TABLE 32.

Velocities and discharges in trapezoidal channels, based on Kutter's formula, with $n = .03$. Side slopes ½ horizontal to 1 vertical.

BED WIDTH 100 FEET.

Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
3.	2443	2	609.
3.	1090	3	913.5
3.	614	4	1218.
3.	393	5	1522.5
3.25	2748	2	660.6
3.25	1223	3	990.8
3.25	688	4	1321.1
3.25	440	5	1651.4
3.5	3029	2	712.3
3.5	1346	3	1068.4
3.5	757	4	1424.5
3.5	485	5	1780.6
4.	3650	2	816.
4.	1614	3	1224.
4.	908	4	1632.
4.	581	5	2040.
4.	4221	2	920.3
4.	1859	3	1380.4
4.	1045	4	1840.5
4.	668	5	2300.6
5.	4913	2	1025.
5.	2161	3	1537.5
5.	1210	4	2050.
5.	774	5	2562.5
6.	6231	2	1236.
6.	2714	3	1854.
6.	1512	4	2472.
6.	963	5	3090.

BED WIDTH 120 FEET.

Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
4.	3652	2	976.
4.	1612	3	1464.
4.	906	4	1952.
4.	580	5	2440.
5.	4989	2	1225.
5.	2190	3	1837.5
5.	1224	4	2450.
5.	784	5	3062.5
6.	6462	2	1476.
6.	2796	3	2214.
6.	1554	4	2952.
6.	986	5	3690.
7.	7914	2	1729.
7.	3389	3	2593.5
7.	1879	4	3458.
7.	1195	5	4322.5
8.	9595	2	1984.
8.	4034	3	2976.
8.	2231	4	3968.
8.	1412	5	4960.

BED WIDTH 140 FEET.

Depth in feet.	Slope 1 in	Velocity in feet per second.	Discharge in cubic feet per second.
4.	3701	2	1136.
4.	1640	3	1704.
4.	921	4	2272.
4.	589	5	2840.
5	5051	2	1425.
5.	2217	3	2137.5
5.	1241	4	2850.
5.	794	5	3562.5
6.	6533	2	1716.
6.	2811	3	2574.
6.	1568	4	3432.
6.	997	5	4290.
7.	8109	2	2009.
7.	3462	3	3013.5
7.	1925	4	4018.
7.	1221	5	5022.5
8.	9795	2	2304.
8.	4116	3	3456.
8.	2278	4	4608.
8.	1443	5	5760.
9.	11453	2	2601.
9.	4822	3	3901.5
9.	2632	4	5202.
9.	1633	5	6502.5

TABLE 33.

Giving fall in feet per mile; the distance on slope corresponding to a fall of one foot, and also the values of s and $\sqrt{s}$.

$s = \frac{h}{l}$ = sine of slope = fall of water surface (h), in any distance (l), divided by that distance.

Fall in inches per mile.	Slope 1 in	s	$\sqrt{s}$	Fall in feet per mile.	Slope 1 in	s	$\sqrt{s}$
2	31680	.000031565	.005618	.25	21120	.000047349	.006881
2½	25344	.000039457	.006281	.50	10560	.000094697	.009731
3½	18103	.000055240	.007432	.75	7040	.000142045	.011918
4	15840	.000063131	.007945	1.	5280	.000189393	.013762
4½	14080	.000071023	.008427	1.25	4224	.000236742	.015386
5	12672	.000078913	.008883	1.5	3520	.000284091	.016854
5½	11520	.000086805	.009317	1.75	3017	.000331439	.018205
6½	9748	.000102588	.010129	2.	2640	.000378788	019463
7	9051	.000110479	.010511	2.25	2347	.000426076	.020641
7½	8448	.000118371	.010880	2.5	2112	.000473485	.021760
8	7920	.000126261	.011237	2.75	1920	.000520833	.022822
8½	7454	.000134154	.011583	3.	1760	.000568182	.023837
9½	6670	.000149937	.012245	3.25	1625	.000615384	.024807
10	6336	.000157828	.012563	3.5	1508	.000663130	.025751
10½	6034	.000165720	.012873	3.75	1408	.000710227	.026650
11	5760	.000173598	.013176	4.	1320	.000757576	.027524
11½	5510	.000181502	.013472	5.	1056	.000946970	.030773
12	5280	.000189393	.013762	6.	880.	.001136364	.03371
12½	5069	.000197285	.014016	7.	754.3	.001325732	.036416
12¾	4969	.000201231	.014185	8.	660.	.001515152	.038925
13	4874	.000205182	.014324	9.	586.6	.001704445	.041286
13½	4693	.000213068	.014597	10.	528.	.001893939	.043519
13¾	4608	.000217014	.014732	11.	443.6	.002083333	.045643
14	4526	000220960	.014865	12.	440.	.002272727	.047673
14⅛	4425	.000225989	.015033	13.	406.1	.002462121	.04962
14½	4370	.000228851	.015128	14.	377.1	.002651515	.051493
14⅚	4271	.000234137	.015301	15.	352.	.002840909	.0533
15½	4088	.000244634	.015641	16.	330.	.003030303	.055048
16	3960	.000252525	.015891	17.	310.6	.003219696	.056742
16½	3840	.000260411	.016137	18.	293.3	.003409090	.058388
17	3727	.000268308	.016381	19.	277.9	.003598484	.059988
17½	3621	.000276199	.016619	20.	264.	.003787878	.061546
18½	3425	.000291982	.017087	21.	251.4	.003977272	.063066
19	3335	.000299874	.017317	22.	240.	.004166667	.064549
19½	3249	.000307765	.017543	23.	229.6	.004356060	.066
20	3168	.000315656	.017767	24.	220.	.004545454	.067419
20½	3091	.000323548	.017987	25	211.2	.004734848	.06881
21½	2947	.000339331	.018421	26.	203.1	.004924242	.070173
22	2880	.000347222	.018634	27.	195.2	.005113636	.07151
22½	2816	.000355114	.018844	28.	188.6	.005303030	.072822
23	2755	.000363005	.019052	29.	182.1	.005492424	.074111
23½	2696	.000370896	.019259	30.	176.	.005681818	.075373

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
4	1320.	.25	.5	51	103.5	.019607843	.140028
5	1056.	.2	.447214	52	101.5	.019230769	.138676
6	880.	.166666666	.408248	53	99.62	.018867925	.137361
7	754.3	.142857143	.377978	54	97.78	.018518519	.136085
8	660.	.125	.353553	55	96.	.018181818	.134839
9	586.7	.111111111	.333333	56	94.29	.017850143	.133630
10	528.	.1	.316228	57	92.65	.017543860	.132453
11	480.	.090909090	.301511	58	91.03	.017241379	.131305
12	440.	.083333333	.288675	59	89.49	.016949153	.130189
13	406.2	.076923077	.277350	60	88.	.016666667	.129100
14	377.1	.071428571	.267261	61	86.56	.016393443	.128037
15	352.	.066666667	.258199	62	85.16	.016129032	.127000
16	330.	.0625	.25	63	83.81	.015873016	.125988
17	310.6	.058823529	.242536	64	82.50	.015625	.125
18	293.3	.055555555	.235702	65	81.23	.015384615	.124035
19	277.9	.052631579	.229416	66	80.	.015151515	.123091
20	264.	.05	.223607	67	78.81	.014925353	.122169
21	251.4	.047619048	.218218	68	77.65	.014705882	.121286
22	240.	.045454545	.213200	69	76.52	.014492754	.120386
23	229.6	.043478261	.208514	70	75.43	.014285714	.119524
24	220.	.041666667	.204124	71	74.36	.014084507	.118678
25	211.2	.04	.2	72	73.33	.013888889	.117851
26	203.1	.038461538	.196116	73	72.33	.013688630	.117041
27	195.6	.037037037	.192450	74	71.35	.013513514	.116248
28	188.6	.035714286	.188982	75	70.40	.013333333	.115470
29	182.1	.034452759	.185695	76	69.47	.013157895	.114708
30	176.	.033333333	.182574	77	68.57	.012987013	.113961
31	170.3	.032258065	.179605	78	67.69	.012820513	.113228
32	165.	.03125	.176777	79	66.84	.012658228	.112509
33	160.	.030303030	.174077	80	66.	.0125	.111803
34	155.3	.029411765	.171499	81	65.18	.012345679	.111111
35	150.9	.028571429	.169031	82	64.39	.012195122	.110431
36	146.7	.027777778	.166667	83	63.62	.012048193	.109764
37	142.7	.027027027	.164399	84	62.86	.011904762	.109109
38	138.9	.026315789	.162221	85	62.12	.011764706	.108465
39	135.4	.025641026	.160125	86	61.40	.011627907	.107833
40	132.	.025	.158114	87	60.69	.011494253	.107211
41	128.8	.024390244	.156174	88	60.	.011363636	.106600
42	125.7	.023809524	.154303	89	59.32	.011235955	.106000
43	122.8	.023255814	.152499	90	58.66	.011111111	.105409
44	120.	.022727273	.150756	91	58.02	.010989011	.104828
45	117.3	.022222222	.149071	92	57.39	.010869565	.104257
46	114.8	.021739130	.147444	93	56.78	.010752688	.103695
47	112.3	.021276600	.145865	94	56.17	.010638298	.103142
48	110.	.020833333	.144337	95	55.58	.010526316	.102598
49	107.8	.020408163	.142857	96	55.	.010416667	.102062
50	105.6	.02	.141421	97	54.43	.010309278	.101535

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
98	53.88	.010204082	.101015	145	36.41	.006896552	.083046
99	53.34	.010101010	.100504	146	36.16	.006849315	.082760
100	52.8	.010	.1	147	35.92	.006802721	.082479
101	52.28	.009900990	.099504	148	35.68	.006756757	.082199
102	51.76	.009803922	.099015	149	35.44	.006711409	.081923
103	51.26	.009708738	.098533	150	35.20	.006666667	.081650
104	50.77	.009615385	.098058	151	34.97	.006622517	.081379
105	50.29	.009523810	.097590	152	34.74	.006578947	.081111
106	49.81	.009433962	.097129	153	34.51	.006535948	.080845
107	49.35	.009345794	.096674	154	34.29	.006493506	.080582
108	48.89	.009259259	.096225	155	34.06	.006451613	.080322
109	48.44	.009174312	.095783	156	33.85	.006410256	.080065
110	48.	.009090909	.095346	157	33.63	.006369427	.079809
111	47.57	.009009009	.094916	158	33.42	.006329114	.079556
112	47.14	.008928571	.094491	159	33.21	.006289308	.079305
113	46.72	.008849558	.094072	160	33.	.00625	.079057
114	46.31	.008771930	.093659	161	32.8	.006211180	.078811
115	45.91	.008695652	.093250	162	32.59	.006172840	.078568
116	45.52	.008620690	.092848	163	32.39	.006134969	.078326
117	45.13	.008547009	.092450	164	32.20	.006097561	.078087
118	44.75	.008474576	.092057	165	32.	.006060606	.077850
119	44.37	.008403361	.091669	166	31.81	.006024096	.077615
120	44.	.008333333	.091287	167	31.62	.005988024	.077382
121	43.64	.008264463	.090909	168	31.43	.005952381	.077152
122	43.28	.008196721	.090536	169	31.24	.005917160	.076923
123	42.93	.008130081	.090167	170	31.06	.005882353	.076697
124	42.58	.008064516	.089803	171	30.88	.005847953	.076472
125	42.24	.008	.089442	172	30.7	.005813953	.076249
126	41.91	.007936508	.089087	173	30.52	.005780347	.076029
127	41.58	.007874016	.088736	174	30.34	.005747126	.075810
128	41.25	.0078125	.088388	175	30.17	.005714286	.075593
129	40.93	.007751938	.088045	176	30.	.005681818	.075378
130	40.62	.007692308	.087706	177	29.83	.005649718	.075164
131	40.31	.007633588	.087370	178	29.66	.005617978	.074953
132	40.	.007575758	.087039	179	29.50	.005586592	.074744
133	39.70	.007518797	.086711	180	29.33	.005555556	.074536
134	39.40	.007462687	.086387	181	29.17	.005524862	.074329
135	39.11	.007407407	.086066	182	29.01	.005494505	.074125
136	38.82	.007352941	.085749	183	28.85	.005464481	.073922
137	38.54	.007299270	.085436	184	28.70	.005434783	.073721
138	38.26	.007246377	.085126	185	28.54	.005405405	.073521
139	37.98	.007194245	.084819	186	28.39	.005376344	.073324
140	37.71	.007142857	.084516	187	28.24	.005347594	.073127
141	37.45	.007092199	.084215	188	28.09	.005319149	.072932
142	37.18	.007042254	.083918	189	27.94	.005291005	.072739
143	36.92	.006993007	.083624	190	27.79	.005263158	.072548
144	36.67	.006944444	.083333	191	27.64	.005235602	.072357

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
192	27.50	.005208333	.072169	395	13.37	.002531646	.050315
193	27.36	.005181347	.071982	400	13.20	.002500000	.050000
194	27.22	.005154639	.071796	405	13.04	.002469136	.049690
195	27.08	.005128205	.071612	410	12.88	.002439024	.049387
196	26.94	.005102041	.071429	415	12.72	.002409639	.049088
197	26.80	.005076142	.071247	420	12.57	.002380952	.048795
198	26.67	.005050505	.071067	425	12.42	.002352941	.048507
199	26.53	.005025126	.070888	430	12.28	.002325581	.048224
200	26.40	.005	.070710	435	12.14	.002298851	.047946
205	25.76	.004878049	.069843	440	12.	.002272727	.047673
210	25.14	.004761905	.069007	445	11.87	.002247191	.047404
215	24.56	.004651163	.068199	450	11.73	.002222222	.047140
220	24.	004545454	.067419	455	11.60	.002197802	.046880
225	23.47	.004444444	.066667	460	11.48	.002173913	.046625
230	22.96	.004347826	.065938	465	11.35	.002150538	.046374
235	22.48	.004255319	.065233	470	11.24	.002127660	.046126
240	22.	.004166667	.064549	475	11.12	.002105263	.045883
245	21.55	.004081623	.063885	480	11.	.002083333	.045644
250	21.12	.004000000	.063246	485	10.89	.002061856	.045407
255	20.71	.003921569	.062620	490	10.78	.002040816	.045175
260	20.31	.003846154	.062018	495	10.67	.002020202	.044947
265	19.92	.003773585	.061430	500	10.56	.002000000	.044721
270	19.56	.003703704	.060858	505	10.46	.001980198	.044499
275	19.20	.003633634	.060302	510	10.35	.001960784	.044281
280	18.86	.003571429	.059761	515	10.25	.001941748	.044065
285	18.53	.003508772	.059235	520	10.15	.001923077	.043853
290	18.20	.003448276	.058722	525	10.06	.001904763	.043644
295	17.90	.003389831	.058222	530	9.962	.001886792	.043437
300	17.60	.003333333	.057735	535	9.870	.001869159	.043234
305	17.31	.003278689	.057260	540	9.778	.001851852	.043033
310	17.03	.003225806	.056796	545	9.688	.001834862	.042835
315	16.76	.003174603	056344	550	9.600	.001818182	.042640
320	16.50	.003125000	.055902	555	9.513	.001801802	.042448
325	16.25	.003076923	.055470	560	9.428	.001785714	.042258
330	16.	.003030303	.055048	565	9.345	.001769912	.042070
335	15.76	.002985075	.054636	570	9.263	.001754386	.041885
340	15.53	.002941176	.054232	575	9.182	.001739130	.041703
345	15.30	.002898551	.053838	580	9.103	.001724138	.041523
350	15.09	.002857143	.053452	585	9.026	.001709420	.041345
355	14.87	.002816901	.053074	590	8.949	.001694915	.041169
360	14.67	.062777778	.052705	595	8.874	.001680672	.040996
365	14.47	.002739726	.052342	600	8.800	.001666667	.040825
370	14.27	.002702703	.051988	605	8.727	.001652893	.040656
375	14.08	.002666667	.051640	610	8.656	.001639344	.040489
380	13.90	.002631579	.051299	615	8.585	.001626016	.040324
385	13.71	.002597403	.050965	620	8.516	.001612903	.040161
390	13.54	.002564103	.050637	625	8.448	.001600000	.040000

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
630	8.381	.001587302	.039841	865	6.104	.001156069	.034001
635	8.317	.001574803	.039684	870	6.069	.001149425	.033903
640	8.250	.001562500	.039528	875	6.034	.001142857	.033806
645	8.186	.001550388	.039375	880	6.	.001136364	.033710
650	8.123	.001538462	.039223	885	5.966	.001129944	.033614
655	8.061	.001526718	.039073	890	5.932	.001123597	.033520
660	8.	.001515152	.038925	895	5.900	.001117318	.033426
665	7.940	.001503759	.038778	900	5.867	.001111111	.033333
670	7.881	.001492537	.038633	905	5.834	.001104972	.033241
675	7.822	.001481481	.038490	910	5.802	.001100110	.033108
680	7.765	.001470588	.038348	915	5.770	.001093896	.033059
685	7.708	.001459854	.038208	920	5.739	.001086957	.032969
690	7.652	.001449275	.038069	925	5.708	.001081081	.032879
695	7.597	.001438849	.037932	930	5.677	.001075269	.032791
700	7.543	.001428571	.037796	935	5.648	.001069519	.032703
705	7.490	.001418440	.037662	940	5.617	.001063830	.032616
710	7.437	.001408451	.037529	945	5.587	.001058201	.032530
715	7.385	.001398601	.037398	950	5.558	.001052632	.032444
720	7.333	.001388889	.037268	955	5.528	.001047120	.032359
725	7.283	.001379310	.037139	960	5.500	.001041667	.032275
730	7.233	.001369863	.037012	965	5.472	.001036269	.032191
735	7.184	.001360544	.036885	970	5.434	.001030928	.032108
740	7.135	.001351351	.036761	975	5.415	.001025641	.032026
745	7.087	.001342282	.036637	980	5.388	.001020408	.031944
750	7.040	.001333333	.036515	985	5.360	.001015228	.031863
755	6.993	.001324503	.036394	990	5.333	.001010101	.031782
760	6.948	.001315789	.036274	995	5.366	.001005025	.031702
765	6.902	.001307190	.036155	1000	5.280	.001000000	.031623
770	6.857	.001298701	.036038	1005	5.253	.000985025	.031544
775	6.812	.001290323	.035921	1010	5.228	.00099099	.031466
780	6.769	.001282051	.035806	1015	5.202	.000985222	.031388
785	6.726	.001273885	.035691	1020	5.176	.000980392	.031311
790	6.684	.001265823	.035578	1025	5.151	.000975610	.031235
795	6.642	.001257862	.035466	1030	5.126	.000970873	.031159
800	6.600	.001250000	.035355	1035	5.101	.000966184	.031083
805	6.559	.001242236	.035245	1040	5.077	.000961538	.031009
810	6.518	.001234568	.035136	1045	5.053	.000956938	.030934
815	6.478	.001226994	.035028	1050	5.029	.000952381	.030861
820	6.439	.001219512	.034922	1055	5.005	.000947867	.030787
825	6.400	.001212121	.034816	1060	4.981	.000943396	.030715
830	6.362	.001204819	.034710	1065	4.958	.000938967	.030643
835	6.324	.001197605	.034606	1070	4.935	.000934579	.030571
840	6.2[illegible]6	.001190476	.034503	1075	4.912	.000930233	.030499
845	6.248	.001183432	.034401	1080	4.889	.000925926	.030429
850	6.212	.001176471	.034300	1085	4.866	.000921659	.030359
855	6.175	.001169591	.034199	1090	4.844	.000917431	.030289
860	6.140	.001162791	.034099	1095	4.822	.000913242	.030220

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
1100	4.800	.000909090	.030151	1335	3.955	.000749064	.027369
1105	4.778	.000904159	.030069	1340	3.940	.000746268	.027318
1110	4.757	.000900900	.030015	1345	3.926	.000743420	.027267
1115	4.735	.000896861	.029948	1350	3.911	.000740741	.027217
1120	4.714	.000892857	.029881	1355	3.897	.000738007	.027166
1125	4.693	.000888888	.029814	1360	3.882	.000735294	.027116
1130	4.673	.000884956	.029748	1365	3.868	.000732601	.027067
1135	4.652	.000881057	.029683	1370	8.854	.000729927	.027017
1140	4.632	.000877193	.029617	1375	3.840	.000727273	.026968
1145	4.611	.000873365	.029553	1380	3.826	.000724638	.026919
1150	4.591	.000869566	.029488	1385	3.812	.000722022	.026870
1155	4.571	.000865801	.029425	1390	3.799	.000719424	.026822
1160	4.552	.000862069	.029361	1395	3.785	.000716846	.026774
1165	4.532	.000858370	.029298	1400	3.771	.000714286	.026726
1170	4.513	.000854701	.029235	1405	3.758	.000711744	.026679
1175	4.494	.000851064	.029173	1410	3.745	.000709220	.026631
1180	4.475	.000847458	.029111	1415	3.731	.000706714	.026584
1185	4.456	.000843882	.029049	1420	3.718	.000704225	.026537
1190	4.437	.000840336	.028988	1425	3.705	.000701754	.026491
1195	4.418	.000836820	.028928	1430	3.692	.000699300	.026444
1200	4.400	.000833333	.028868	1435	3.680	.000696864	.026398
1205	4.382	.000829875	.028808	1440	3.667	.000694444	.026352
1210	4.364	.000826446	.028748	1445	3.654	.000692042	.026307
1215	4.346	.000823045	.028689	1450	3.641	.000689655	.026261
1220	4.328	.000819672	.028630	1455	3.629	.000687285	.026216
1225	4.310	.000816326	.028571	1460	3.617	.000684931	.026171
1230	4.293	.000813008	.028513	1465	3.604	.000682594	.026126
1235	4.275	.000809717	.028455	1470	3.592	.000680272	.026082
1240	4.258	.000806452	.028398	1475	3.580	.000677966	.026038
1245	4.241	.000803213	.028341	1480	3.568	.000675676	.025994
1250	4.224	.000800000	.028284	1485	3.556	.000673401	.025950
1255	4.207	.000796813	.028228	1490	3.544	.000671141	.025907
1260	4.190	.000793651	.028172	1495	3.532	.000668896	.025863
1265	4.174	.000790514	.028116	1500	3.520	.000666666	.025820
1270	4.157	.000787402	.028061	1505	3.508	.000664452	.025777
1275	4.141	.000784314	.028006	1510	3.497	.000662252	.025734
1280	4.125	.000781250	.027951	1515	3.485	.000660066	.025691
1285	4.109	.000778210	.027896	1520	3.474	.000657895	.025649
1290	4.093	.000775116	.027841	1525	3.462	.000655737	.025607
1295	4.077	.000772201	.027789	1530	3.451	.000653595	.025566
1300	4.062	.000769231	.027735	1535	3.440	.000652117	.025524
1305	4.046	.000766283	.027682	1540	3.429	.000649351	.025482
1310	4.031	.000763359	.027629	1545	3.417	.000647275	.025441
1315	4.015	.000760456	.027576	1550	3.407	.000645161	.025400
1320	4.	.000757576	.027524	1555	3.396	.000643087	.025359
1325	3.985	.000754717	.027472	1560	3.385	.000641025	.025318
1330	3.970	.000751880	.027420	1565	3.374	.000638978	.025278

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
1570	3.363	.000636943	.025238	1805	2.925	.000554017	.023538
1575	3.352	.000634921	.025198	1810	2.917	.000552486	.023505
1580	3.342	.000632911	.025158	1815	2.909	.000550964	.023473
1585	3.331	.000630915	.025118	1820	2.901	.000549451	.023440
1590	3.321	.000628931	.025078	1825	2.893	.000547945	.023408
1595	3.310	.000626959	.025039	1830	2.885	.000546448	.023376
1600	3.300	.000625000	.025000	1835	2.877	.000544949	.023344
1605	3.290	.000623053	.024961	1840	2.870	.000543478	.023313
1610	3.280	.000621118	.024922	1845	2.862	.000542005	.023281
1615	3.269	.000619195	.024884	1850	2.854	.000540541	.023250
1620	3.259	.000617284	.024845	1855	2.847	.000539084	.023218
1625	3.249	.000615384	.024807	1860	2.839	.000537633	.023187
1630	3.239	.000613497	.024769.	1865	2.831	.000536193	.023156
1635	3.229	.000611621	.024731	1870	2.824	.000534759	.023125
1640	3.220	.000699756	.024693	1875	2.816	.000533333	.023094
1645	3.210	.000607900	.024656	1880	2.809	.000531915	.023063
1650	3.200	.000606060	.024618	1885	2.801	.000530504	.023033
1655	3.190	.000604230	.024581	1890	2.794	.000529101	.023002
1660	3.181	.000602409	.024544	1895	2.786	.000527705	.022972
1665	3.171	.000600601	.024507	1900	2.779	.000526316	.022942
1670	3.162	.000598802	.024470	1905	2.772	.000524934	.022911
1675	3.152	.000597015	.024434	1910	2.764	.000523560	.022881
1680	3.143	.000595238	.024398	1915	2.757	.000522193	.022852
1685	3.134	.000593102	.024354	1920	2.750	.000520833	.022822
1690	3.124	.000591717	.024325	1925	2.743	.000519481	.022792
1695	3.115	.000589971	.024290	1930	2.736	.000518135	.022763
1700	3 106	.000588235	.024254	1935	2.729	.000516796	.022733
1705	3.097	.009586510	.024218	1940	2.722	.000515464	.022704
1710	3.088	.000584795	.024183	1945	2.715	.000514139	.022675
1715	3.079	.000583090	.024147	1950	2.708	.000512821	.022646
1720	3.070	.000581395	.024112	1955	2.701	.000511509	.022616
1725	3.061	.000579710	.024077	1960	2.694	.000510204	.022588
1730	3.052	.000578035	.024042	1965	2.687	.000508906	.022559
1735	3.042	.000576369	.024008	1970	2.680	.000507614	.022530
1740	3.035	.000574712	.023973	1975	2.673	.000506329	.022502
1745	3.026	.000573066	.023939	1980	2.667	.000505051	.022473
1750	3.017	.000571429	.023905	1985	2.660	.000503778	.022445
1755	3.009	.000569801	.023871	1990	2.653	.000502513	.022417
1760	3.	.000568182	.023837	1995	2.647	.000501253	.022388
1755	2.992	.000566572	.023803	2000	2.640	.000500000	.022361
1770	2.983	.000564972	.023769	2005	2.633	.000498753	.022333
1775	2.975	.000563380	.023736	2010	2.627	.000497512	.022305
1780	2.966	.000561798	.023702	2015	2.620	.000496278	.022277
1785	2.958	.000560224	.023669	2020	2.614	.000495050	.022250
1790	2.950	.000558659	.023636	2025	2.607	.000493827	.022222
1795	2.942	.000557103	.023603	2030	2.601	.000492611	.022195
1800	2.933	.000555555	.023570	2035	2.595	.000491400	.022168

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
2040	2.588	.000490196	.022140	2265	2.331	.000441501	.021012
2045	2.582	.000488998	.022113	2270	2.326	.000440529	.020989
2050	2.576	.000487805	.022086	2275	2.321	.000439560	.020966
2055	2.569	.000486618	.022059	2280	2.316	.000438597	.020943
2060	2.563	.000485437	.022033	2285	2.311	.000437637	.020920
2065	2.557	.000484213	.022005	2290	2.306	.000436681	.020897
2070	2.551	.000483093	.021979	2295	2.301	.000435730	.020874
2075	2.545	.000481928	.021953	2300	2.296	.000434783	.020853
2080	2.538	.000480769	.021926	2305	2.291	.000433839	.020829
2085	2.532	.000479616	.021900	2310	2.286	.000432900	.020806
2090	2.526	.000478469	.021874	2315	2.281	.000431965	.020784
2095	2.520	.000477327	.021848	2320	2.276	.000431034	.020761
2100	2.514	.000476190	.021822	2325	2.271	.000430108	.020740
2105	2.508	.000475059	.021796	2330	2.266	.000429185	.020717
2110	2.502	.000473934	.021770	2335	2.261	.000428266	.020694
2115	2.496	.000472813	.021744	2340	2.256	.000427350	.020672
2120	2.491	.000471698	.021719	2345	2.252	.000426439	.020650
2125	2.485	.000470588	.021693	2350	2.247	.000425532	.020628
2130	2.479	.000469484	.021668	2355	2.242	.000424629	.020607
2135	2.473	.000468384	.021642	2360	2.237	.000423729	.020585
2140	2.467	.000467290	.021617	2365	2.233	.000422833	.020563
2145	2.462	.000466200	.021592	2370	2.228	.000421941	.020541
2150	2.456	.000465116	.021567	2375	2.223	.000421053	.020520
2155	2.450	.000464037	.021542	2380	2.219	.000420168	.020498
2160	2.444	.000462963	.021517	2385	2.214	.000419287	.020477
2165	2.439	.000461894	.021492	2390	2.209	.000418410	.020455
2170	2.433	.000460829	.021467	2395	2.205	.000417534	.020434
2175	2.428	.000459770	.021442	2400	2.200	.000416667	.020412
2180	2.422	.000458716	.021418	2405	2.195	.000415801	.020391
2185	2.416	.000457666	.021393	2410	2.191	.000414938	.020370
2190	2.411	.000456621	.021369	2415	2.186	.000414079	.020349
2195	2.405	.000455581	.021344	2420	2.182	.000413223	.020328
2200	2.400	.000454545	.021320	2425	2.177	.000412371	.020307
2205	2.395	.000453515	.021296	2430	2.173	.000411523	.020286
2210	2.389	.000452489	.021272	2435	2.168	.000410678	.020265
2215	2.384	.000451467	.021248	2440	2.164	.000409836	.020244
2220	2.378	.000450450	.021224	2445	2.160	.500408998	.020224
2225	2.373	.000449438	.021200	2450	2.155	.000408163	.020203
2230	2.368	.000448430	.021176	2455	2.151	.000407332	.020182
2235	2.362	.000447427	.021152	2460	2.146	.000406504	.020162
2240	2.357	.000446429	.021129	2465	2.142	.000405680	.020141
2245	2.352	.000445434	.021105	2470	2.138	.000404858	.020121
2250	2.347	.000444444	.021082	2475	2.133	.000404040	.020101
2255	2.341	.000443459	.021058	2480	2.129	.000403226	.020080
2260	2.336	.000442478	.021035	2485	2.125	.000402414	.020060

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
2490	2.120	.000401606	.020040	2715	1.945	.000368324	.019192
2495	2.116	.000400802	.020020	2720	1.941	.000367647	.019174
2500	2.112	.000400000	.020000	2725	1.938	.000366972	.019156
2505	2.108	.000399202	.019980	2730	1.934	.000366300	.019139
2510	2.104	.000398406	.019960	2735	1.931	.000365631	.019121
2515	2.099	.000397614	.019940	2740	1.927	.000364964	.019104
2520	2.095	.000396825	.019920	2745	1.923	.000364299	.019086
2525	2.091	.000396039	.019901	2750	1.920	.000363636	.019069
2530	2.087	.000395257	.019881	2755	1.916	.000362972	.019052
2535	2.083	.000394477	.019861	2760	1.913	.000362319	.019035
2540	2.079	.000393701	.019842	2765	1.910	.000361664	.019017
2545	2.075	.000392927	.019822	2770	1.906	.000361011	.019000
2550	2.071	.000392157	.019803	2775	1.903	.000360360	.018983
2555	2.066	.000391389	.019784	2780	1.900	.000359712	.018966
2560	2.063	.000390625	.019764	2785	1.896	.000359066	.018949
2565	2.058	.000389864	.019745	2790	1.892	.000358423	.018932
2570	2.054	.000389105	.019726	2795	1.889	.000357782	.018915
2575	2.050	.000388349	.019706	2800	1.886	.000357143	.018898
2580	2.047	.000387697	.019687	2805	1.882	.000356506	.018881
2585	2.042	.000386847	.019668	2810	1.879	.000355871	.018865
2590	2.039	.000386100	.019649	2815	1.875	.000355279	.018848
2595	2.035	.000385357	.019630	2820	1.872	.000354610	.018831
2600	2.031	.000384615	.019612	2825	1.869	.000353982	.018814
2605	2.027	.000383877	.019593	2830	1.866	.000353357	.018797
2610	2.023	.000383142	.019574	2835	1.862	.000352733	.018781
2615	2.019	.000382410	.019555	2840	1.859	.000352113	.018764
2620	2.015	.000381679	.019536	2845	1.856	.000351423	.018746
2625	2.011	.000380952	.019518	2850	1.852	.000350877	.018731
2630	2.008	.000380228	.019499	2855	1.849	.000350877	.018715
2635	2.004	.000379507	.019481	2860	1.846	.000349650	.018699
2640	2.	.000378787	.019462	2865	1.843	.000349040	.018682
2645	1.996	.000378072	.019444	2870	1.839	.000348432	.018666
2650	1.992	.000377359	.019426	2875	1.836	.000347827	.018650
2655	1.989	.000376648	.019407	2880	1.833	.000347222	.018634
2660	1.985	.000375940	.019389	2885	1.830	.000346662	.018617
2665	1.981	.000375235	.019371	2890	1.827	.000346021	.018602
2670	1.977	.000374532	.019353	2895	1.824	.000345427	.018585
2675	1.974	.000373832	.019334	2900	1.820	.000344827	.018569
2680	1.970	.000373134	.019316	2905	1.817	.000344234	.018554
2685	1.966	.000372437	.019298	2910	1.814	.000343643	018537
2690	1.963	.000371747	.019281	2915	1.811	.000343057	.018521
2695	1.959	.000371058	.019263	2920	1.808	.000342456	.018506
2700	1.956	.000370370	.019245	2925	1.805	.900341880	.018490
2705	1.952	.000369686	.019228	2930	1.802	.000341297	.018474
2710	1.949	.000369004	.019209	2935	1.799	.000340716	.018456

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
2940	1.796	.000340136	.018442	3460	1.526	.000289017	.017000
2945	1.793	.000339559	.018427	3480	1.517	.000287356	.016951
2950	1.790	.000338983	.018414	3500	1.509	.000285714	.016903
2955	1.787	.000338409	.018396	3520	1.500	.000284091	.016855
2960	1.784	.000337838	.018380	3540	1.491	.000282486	.016807
2965	1.781	.000337268	.018264	3560	1.483	.000280899	.016760
2970	1.778	.000336700	.018349	3580	1.475	.000279329	.016713
2975	1.775	.000336134	.018334	3600	1.467	.000277778	.016667
2980	1.772	.000335571	.018319	3620	1.459	.000276243	.016620
2985	1.769	.060335008	.018303	3640	1.450	.000274725	.016575
2990	1.766	.000334482	.018288	3660	1.442	.000273224	.016530
2995	1.763	.000333890	.018272	3680	1.435	.000271739	.016484
3000	1.760	.000333333	.018257	3700	1.427	.000270270	.016440
3010	1.754	.000332226	.018227	3720	1.420	.000268817	.016395
3020	1.748	.000331129	.018197	3740	1.412	.000267380	.016352
3030	1.742	.000330033	.018667	3760	1.404	.000265958	.016308
3040	1.737	.000328947	.018137	3780	1.397	.000264550	.016265
3050	1.731	.000327869	.018107	3800	1.390	.000263158	.016222
3060	1.725	.000326797	.018077	3820	1.382	.000261780	.016180
3070	1.720	.000325733	.018048	3840	1.375	.000260417	.016138
3080	1.715	.000324675	.018019	3860	1.368	.000259067	.016095
3090	1.709	.000323625	.017989	3880	1.361	.000257732	.016054
3100	1.703	.000322581	.017960	3900	1.354	.000256410	.016013
3110	1.698	.000321543	.017932	3920	1.347	.000255102	.015972
3120	1.692	.000320513	.017903	3940	1.340	.000253807	.015931
3130	1.687	.000319489	.017874	3960	1.333	.000252525	.015891
3140	1.682	.000318471	.017845	3980	1.327	.000251256	.015851
3150	1.676	.000317460	.017817	4000	1.320	.000250000	015811
3160	1.671	.000316456	.017789	4020	1.313	.000248756	.015772
3170	1.666	.000315457	.017761	4040	1.307	.000247525	.015733
3180	1.660	.000314465	.017733	4060	1.300	.000246306	.015694
3190	1.655	.000313480	.017705	4080	1.294	.000245098	.015655
3200	1.650	.000312500	.017677	4100	1.288	.000243903	.015617
3220	1.640	000310559	.017622	4120	1.282	.000242718	.015580
3240	1.629	000308641	.017568	4140	1.275	.000241546	.015542
3260	1.620	.000306748	.017514	4160	1.269	.000240382	.015505
3280	1.610	.000304878	.017461	4180	1.263	.000239235	.015467
3300	1.600	.000303030	.017408	4200	1.257	.000238095	.015430
3320	1.590	.000301205	.017355	4220	1.251	.000236967	.015394
3340	1.581	.000299401	.017303	4240	1.245	.000235849	.015358
3360	1.571	.000297619	.017251	4260	1.239	.000234742	.015322
3380	1.562	.000295858	.017200	4280	1.234	.000233645	.015286
3400	1.553	.000294118	.017150	4300	1.228	.000232558	.015250
3420	1.544	.000292398	.017100	4320	1.222	.000231482	.015215
3440	1.535	.000290688	.017050	4340	1.217	.000230415	.015180

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
4360	1.211	.000229716	.015145	6000	.880	.000166667	.012910
4380	1.205	.000228311	.015110	6080	.868	.000164474	.012820
4400	1.200	.000227273	.015076	6160	.857	.000162338	.012741
4420	1.194	.000226244	.015041	6240	.846	.000160256	.012659
4440	1.189	.000225225	.015007	6320	.836	.000158228	.012579
4460	1.184	.000224215	.014974	6400	.825	.000156250	.012500
4480	1.179	.000223214	.014940	6480	.815	.000154321	.012422
4500	1.173	.000222222	.014907	6560	.805	.000152439	.012347
4520	1.168	.000221239	.014874	6640	.795	.000150602	.012272
4540	1.163	.000220264	.014841	6720	.786	.000148810	.012199
4560	1.158	.000219298	.014808	6800	.777	.000147059	.012127
4580	1.153	.000218341	.014776	6880	.767	.000145349	.012056
4600	1.148	.000217391	.014744	6960	.759	.000143678	.011986
4620	1.143	.000216450	.014712	7000	.754	.000142857	.011952
4640	1.138	.000215517	.014681	7040	.750	.000142045	.011919
4660	1.133	.000214592	.014649	7120	.742	.000140449	.011851
4680	1.128	.000213675	.014617	7200	.733	.000138889	.011785
4700	1.124	.000212766	.014586	7280	.725	.000137363	.011720
4720	1.119	.000211864	.014557	7360	.718	.000135869	.011656
4740	1.114	.000210970	.014524	7440	.710	.000134408	.011594
4760	1.109	.000210084	.014492	7500	.704	.000133333	.011547
4780	1.104	.000209205	.014464	7520	.702	.000132979	.011532
4800	1.100	.000208333	.014434	7600	.695	.000131579	.011471
4820	1.096	.000207469	.014404	7680	.687	.000130208	.011411
4840	1.091	.000206612	.014374	7760	.680	.000128866	.011352
4860	1.087	.000205761	.014344	7840	.673	.000127551	.011293
4880	1.082	.000204918	.014315	7920	.667	.000126263	.011237
4900	1.078	.000204081	.014285	8000	.660	.000125000	.011180
4920	1.073	.000203252	.014256	8080	.653	.000123763	.011125
4940	1.069	.000202429	.014227	8160	.647	.000122549	.011070
4960	1.065	.000201613	.014199	8240	.641	.000121359	.011016
4980	1.060	.000200803	.014170	8320	.635	.000120192	.010963
5000	1.056	.000200000	.014142	8400	.629	.000119048	.010911
5040	1.048	.000198570	.014086	8480	.623	.000117925	.010860
5120	1.031	.000195313	.013975	8560	.617	.000116823	.010809
5200	1.015	.000192308	.013888	8640	.611	.000115741	.010759
5280	1.	.000189394	.013862	8720	.605	.000114679	.010709
5360	.985	.000186567	.013659	8800	.600	.000113636	.010660
5440	.971	.000183824	.013558	8880	.595	.000112613	.010612
5520	.957	.000181160	.013460	8960	.585	.000111607	.010565
5600	.943	.000178572	.013363	9000	.587	.000111111	.010541
5680	.930	.000176056	.013268	9040	.584	.000110620	.010518
5760	.917	.000173611	.013176	9120	.579	.000109649	.010472
5840	.904	.000171233	.013085	9200	.574	.000108696	.010427
5920	.892	.000168919	.012997	9280	.569	.000107759	.010380

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
9360	.564	.000106838	.010336	12880	.410	.000077640	.008811
9440	.559	.000105932	.010293	12960	.407	.000077160	.008784
9520	.555	.000105042	.010249	13000	.406	.000076923	.008771
9600	.550	.000104167	.010206	13040	.405	.000076687	.008757
9680	.545	.000103306	.010164	13120	.402	.000076220	.008730
9760	.541	.000102459	.010122	13200	.400	.000075758	.008704
9840	.537	.000101626	.010081	13280	.398	.000075301	.008678
9920	.532	.000100807	.010040	13360	.395	.000074850	.008651
10000	.528	.000100000	.010000	13440	.393	.000074405	.008625
10080	.524	.000099206	.009960	13520	.390	.000073965	.008600
10160	.520	.000098425	.009921	13600	.388	.000073530	.008575
10240	.516	.000097656	.009882	13680	.386	.000073100	.008550
10320	.512	.000096924	.009844	13750	.384	.000072675	.008525
10400	.508	.000096154	.009806	13840	.382	.000072254	.008500
10480	.504	.000095420	.009768	13920	.379	.000071839	.008476
10560	.500	.000094697	.009731	14000	.377	.000071429	.008452
10640	.496	.000093985	.009695	14080	.375	.000071023	.008428
10720	.492	.000093284	.009658	14160	.373	.000070622	.008404
10800	.489	.000092593	.009623	14240	.371	.000070225	.008380
10880	.485	.000091912	.009587	14320	.369	.000069832	.008357
10960	.482	.000091241	.009552	14400	.367	.000069445	.008334
11000	.480	.000090909	.009534	14480	.365	.000069061	.008310
11040	.478	.000090580	.009518	14560	.363	.000068681	.008288
11120	.475	.000089928	.009483	14640	.361	.000068306	.008265
11200	.471	.000089286	.009449	14720	.359	.000067935	.008242
11280	.468	.000088653	.009416	14800	.357	.000067568	.008220
11360	.465	.000088028	.009382	14880	.355	.000067204	.008198
11440	.462	.000087412	.009350	14960	.353	.000066848	.008176
11520	.458	.000086806	.009317	15000	.352	.000066667	.008165
11600	.455	.000086207	.009285	15040	.351	.000066490	.008154
11680	.452	.000085617	.009253	15120	.349	.000066138	.008133
11760	.449	.000085034	.009221	15200	.347	.000065790	.008111
11840	.446	.000084459	.009190	15280	.346	.000065445	.008090
11920	.443	.000083893	.009160	15360	.344	.000065104	.008069
12000	.440	.000083333	.009129	15440	.342	.000064767	.008048
12080	.437	.000082782	.009099	15520	.340	.000064433	.008027
12160	.434	.000082237	.009069	15600	.339	.000064103	.008007
12240	.431	.000081699	.009039	15680	.337	.000063776	.007986
12320	.429	.000081169	.009010	15760	.335	.000063452	.007966
12400	.426	.000080645	.008980	15840	.333	.000063131	.007946
12480	.423	.000080128	.008951	15920	.332	.000062814	.007926
12560	.420	.000079618	.008923	16000	.330	.000062500	.007906
12640	.418	.000079114	.008895	16080	.328	.000062189	.007886
12720	.415	.000078616	.008867	16160	.327	.000061881	.007867
12800	.413	.000078125	.008839	16240	.325	.000061577	007847

TABLE 33.—SLOPES.

Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$	Slope 1 in	Fall in feet per mile.	s	$\sqrt{s}$
16320	.324	.000061275	.007828	18800	.281	.000053191	.007293
16400	.322	.000060976	.007809	18880	.280	.000052966	.007278
16480	.320	.000060680	.007790	18960	.279	.000052742	.007262
16560	.319	.000060387	.007771	19000	.280	.000052632	.007255
16640	.317	.000060096	.007753	19040	.277	.000052521	.007246
16720	.316	.000059809	.007734	19120	.276	.000052301	.007232
16800	.314	.000059524	.007715	19200	.275	.000052083	.007217
16880	.313	.000059242	.007697	19280	.274	000051867	.007202
16960	.311	.000058962	.007679	19360	.273	.000051653	.007187
17000	.311	.000058824	.007670	19440	.272	.000051440	.007172
17040	.310	.000058686	.007661	19520	.271	.000051229	.007157
17120	.308	.000058411	.007643	19600	.269	.000051020	.007142
17200	.307	.000058140	.007625	19680	.268	.000050813	.007128
17280	.306	.000058146	.007608	19760	.267	.000050607	.007114
17360	.304	.000057604	.007590	19840	.266	.000050403	.007100
17440	.303	.000057429	.007573	19920	.265	.000050201	.007085
17520	.301	.000057078	.007555	20000	.264	.000050000	.007071
17600	.300	.000056818	.007538	20080	.263	.000049800	.007057
17680	.299	.000056561	.007520	20160	.262	.000049603	.007043
17760	.297	.000056306	.007504	20240	.261	.000049407	.007029
17840	.296	.000056054	.007487	20320	.260	.000049212	.007015
17920	.295	.000055804	.007470	20400	.259	.000049020	.007001
18000	.293	.000055555	.007454	20480	.258	.000048828	.006987
18080	.292	.000055310	.007437	20560	.257	.000048638	.006974
18160	.291	.000055066	.007421	20640	.256	.000048447	.006960
18240	.289	.000054825	.007404	20720	.255	.000048263	.006947
18320	.288	.000054585	.007388	20800	.254	.000048077	.006934
18400	.287	.000054348	.007372	20880	.253	.000047893	.006920
18480	.286	.000054112	.007356	20960	.252	.000047710	.006907
18560	.285	000053879	.007340	21040	.251	.000047529	.006894
18640	.283	.000053648	.007324	21120	.250	.000047348	.006881
18720	.282	.000053419	.007308				

Article 14. Formulæ for Mean Velocity in Pipes, Sewers, Conduits, etc.

In continuation of the formulæ for mean velocity in open channels, given at page 8, the following collection of formulæ is given for finding the mean velocity in pipes, sewers, conduits, etc. As already stated, it is believed that such a collection will be useful, not only for reference, but also for comparison with the most modern and accurate formulæ. This list contains almost all the formulæ in use in different countries, in modern times, and it is the most complete collection of formulæ, relating to the *Flow of Water in Open and Closed Channels*, ever before gathered together in a single work.

Some of the formulæ for open channels, already given, have also been used for pipes, sewers, conduits, etc. These will not be reproduced here. They will, however, be denoted by the numbers already given to them. The same symbols are used here as already given at page 6. We have also, in addition:—

d = diameter of pipe in feet, if not otherwise stated.

The formulæ already used for open channels, and which have also been used for pipes, sewers conduits, etc., are:—

D'Aubisson's, Taylor's, Downing's, Beardmore's, Leslie's, Pole's, formula (1); D'Aubisson's (5); Beardmore's (7); Eytelwein's (8); Neville's (12); Dwyer's (13); Young's (16); Dubuat's (17); De Prony's (21); St. Venant's (23); Provis's (25); Fanning's (28); Kutter's (40).

The following formulæ are also applicable to pipes, sewers and conduits:—

D'Arcy's formula for clean iron pipes under pressure is:—

$$v = \left\{ \frac{rs}{.00007726 + \dfrac{.00000162}{r}} \right\}^{\frac{1}{2}} \dots\dots\dots\dots (51)$$

Flynn's modification of D'Arcy's formula is:—

$$v = \left(\frac{155256}{12\,d + 1} \right)^{\frac{1}{2}} \times \sqrt{rs} \dots\dots\dots\dots (52)$$

D'Arcy's formula as given by J. B. Francis, C. E., for old cast-iron pipe, lined with deposit, and under pressure is:—

$$v = \left(\frac{144\,d^2\,s}{.0082\,(12\,d + 1)} \right)^{\frac{1}{2}} \dots\dots\dots\dots (53)$$

Flynn's modification of D'Arcy's formula for old cast-iron pipe is:—

$$v = \left(\frac{70243.9}{12\,d + 1} \right) \times \sqrt{rs} \dots\dots\dots\dots (54)$$

Molesworth's modification of Kutter's formula (40) with $n = .013$ is:—

$$v = \frac{181 + \dfrac{.00281}{s}}{1 + \dfrac{.026}{\sqrt{d}}\left(41.6 + \dfrac{.00281}{s}\right)} \times \sqrt{rs} \dots\dots (55)$$

Flynn's modification of Kutter's formula is (see Article 20, in which are given values of K and $\sqrt{r}$):—

$$v = \left\{ \frac{K}{1 + \left(44.41 \times \dfrac{n}{\sqrt{r}}\right)} \right\} \times \sqrt{rs} \dots\dots (56)$$

Lampe's formula is:—

$$v = 203.3\, r^{.694} \times s^{.555} \dots\dots\dots\dots (57)$$

Weisbach's formula is:—

$$v = \left\{ \frac{2gh}{1.505 + c \times \frac{l}{d}} \right\}^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (58)$$

$$\text{where } c = .01439 + \frac{.016921}{v^{\frac{1}{2}}}$$

Prony's formula is:—

$$v = 97\sqrt{rs} - .08 \text{ nearly} \ldots\ldots\ldots\ldots\ldots (59)$$

Eytelwein's formula is:—

$$v = 108\sqrt{rs} - .13 \text{ nearly} \ldots\ldots\ldots\ldots\ldots (60)$$

Another formula of Eytelwein is:—

$$v = 50\left(\frac{dh}{l + 50d}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (61)$$

D'Aubisson's formula is:—

$$v = 98\sqrt{rs} - .1 \ldots\ldots\ldots\ldots\ldots (62)$$

Hawksley's formula is:—

$$v = 48.05\left(\frac{dh}{l + 54d}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (63)$$

Poncelet's formula is:—

$$v = 47.95\left(\frac{dh}{l + 54d}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (64)$$

Blackwell's formula is:—

$$v = 47.913\left(\frac{dh}{l}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (65)$$

Neville's formula is:—

$$v = \left(\frac{hr}{.0234r + .0001085\,l}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (66)$$

Hughes' modification of Eytelwein's formula (61) is:—

$$v = \left(\frac{df}{2.112}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots (67)$$

Blackwell's modification of Eytelwein's formula (61) is:—

$$v = \left(\frac{df}{2.3}\right)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots \quad (68)$$

Kirkwood's formula for tuberculated pipes is:—

$$v = 80\sqrt{rs} \quad \ldots\ldots\ldots\ldots \quad (69$$

Article 15. Remarks on the Formulæ.

For the purpose of comparison, the formula of D'Arcy and Lampe, for the diameters given in Table 34, have been changed into the form:—

$$v = c\sqrt{rs}$$

and the values of c are given in Table 34.

For the same purpose of comparison the formulæ of Kutter (40), is given in the same table. Kirkwood's formula (69), also given, is modern, but it has a *constant* co-efficient. Also three of the old formulæ are given, namely, Blackwell's (65), Prony's (59), and Downing's (1).

Almost all the old formulæ have *constant* co-efficients. It was well known to many engineers, that these co-efficients gave too high a velocity for small channels, and too low a velocity for large channels. To remedy this, Leslie (see page 8), gave a co-efficient of 100, formula (1), for large and rapid rivers, and a co-efficient of 68, formula (2), for small streams. In the same way Stevenson gave a co-efficient of 96, formula (3), for streams discharging over 2,000 cubic feet per minute, and 69, formula (4), for streams discharging under 2,000 cubic feet per minute. There was no easy curve from one co-efficient to another. It was a sudden increase. It is evident that this cannot be correct. An inspection of the old formulæ will show that their co-

efficients were *constant*, and, according to the different authorities, varied from 92.3 to 100.

The modern and more accurate formulæ have *varying* co-efficients, whose value increases with the increase of the hydraulic mean depth, r.

The value of the co-efficient in D'Arcy's formula (51), depends on the hydraulic mean depth, r, and is not affected by the slope; and it is the same with Lampe's formula (57).

In Kutter's formula (40), the co-efficient depends not only on the hydraulic mean depth, r, but also, to a less extent, on the slope, s.

The co-efficients of the modern formulæ increase very much from the small diameters to the large ones, whereas, the old formulæ have the same co-efficients for all diameters, being too high for diameters under one foot, and too low for diameters exceeding one foot. For diameters larger than 6 feet there is very little change in D'Arcy's co-efficient, and for very large pipes it does not exceed 113.8.

For diameters greater that 10 feet D'Arcy's co-efficient is almost constant. It increases very little more than 113.5, even for a diameter of 16 feet or more, but Kutter's co-efficient continues to increase until such a diameter is reached as is never likely to be required in practice.

Now, the experiments on which D'Arcy's formula is based were made on clean pipes, of the diameters usually adopted in practice, flowing under pressure, and under conditions somewhat similar to pipes in actual use, and, therefore, as the experiments were conducted with great accuracy, the results are entitled to the confidence of engineers. D'Arcy's experiments did not, however, include pipes of a very large hydraulic mean radius. In one respect he differs from most of the mod-

ern authorities, inasmuch as the slope has no effect on the value of the co-efficient of his formula.

Kutter's formula is derived, not only from experiments made on channels with small hydraulic radius, but also on channels with large hydraulic radius, and his co-efficients for very large pipes are, therefore, more likely to agree with the actual discharge than D'Arcy's constant co-efficient of 113.5 for very large pipes. But again, Kutter's formula is open to the objection that it is based on experiments made on open channels. I may here remark, although it is only remotely connected with pipe discharge, that Major Allan Cunningham states, as the result of his extensive experiments for four years on the Ganges Canal, that Kutter's formula alone, of all those tried by him, was found generally applicable to all conditions of discharge, and that it gave nearer results to the actual velocity than any of the other formulæ tried by him. It gave results with a difference from the actual velocity seldom exceeding 5 per cent., and usually much less than that. When we contrast the wide divergence of the old formulæ under varying flow from the actual velocity, with the results obtained by Kutter's formula, it will be seen that the latter is the most accurate formula for channels with large hydraulic mean radius.

With reference to D'Arcy's co-efficients not being affected by the slope, Neville states:—

"As long as the diameter of a long pipe continues constant, the velocity (by D'Arcy's formula) is always represented by a given fixed multiple of $\sqrt{rs}$, no matter how small or great the declivity of the pipe may be. For an inch pipe this multiplier for feet measures is 80.3. * * * * * *

"In the excerpt proceedings of the Institution of Civil Engineers, p. 4, 6th Feb., 1855, James Simpson, Presi-

dent, in the chair, there is given for the "Colinton pipe" 16 inches diameter, eight or nine years in use, three observations.

First, 29,580 feet long, a head of 420 feet, and a discharge of 571 cubic feet per minute. These give $v =$ 6.816 feet $= 99.2 \sqrt{rs}$ nearly. Secondly, a length of 25,765 feet, a head of 184 feet, and a discharge of 440 cubic feet per minute; these give $v = 5.252$ feet $= 96.3 \sqrt{rs}$. And thirdly, a length of 3.815 feet, a head of 184 feet, and a discharge of 1.215 cubic feet per minute; these give $v = 14.5$ feet $= 115 \sqrt{rs}$ nearly. In these three examples the diameter, castings and age of the pipes, are the same. Yet it is seen, clearly, that the inclination affects the multiplier of $\sqrt{rs}$ which increases with the inclination, s, although M. D'Arcy's formula would make the multiplier the same in each case, and for all inclinations, viz.: $v = 110 \sqrt{rs}$."

In the formulæ of Lampe and Kutter the co-efficients have a steady increase with the increase of the diameter.

Kutter's formula has the great advantage of being easily adapted to a change in the surface of the pipe exposed to the flow of water, by a change in the value of n. It will be seen that the co-efficients of Lampe agree somewhat with Kutter with $n = .011$. Now, very few engineers, even with the smoothest pipe, use Kutter with $n = .011$. It is more usual to use $n = .013$, to provide for the future deterioration of the surface exposed to the flow of water.

The 48-inch Glasgow water pipes mentioned at page 218 gave at first a discharge more than that given by the old formulæ, but it gradually diminished, though the pipes still continued to discharge more than the quantity given by the old formulæ.

An inspection of Table 34 will show that for all

diameters greater than 1 foot 6 inches, Lampe's co-efficients are very much greater than D'Arcy's, for clean pipes, and than Kutter with $n = .013$. It is, therefore, evident that, for old pipe, Lampe's formula gives too high a discharge.

The 48-inch pipe given as an example at page 234 has, by D'Arcy's formula for clean pipes (52), a co-efficient $= 112.6$, and in Table 34 we find that for this pipe, Kutter, with $n = .013$, has a co-efficient of 116.5. As the pipe gradually deteriorated D'Arcy's co-efficient 112.6, represented the maximum flow. For this pipe Lampe gives a co-efficient $= 139.0$, being sixteen per cent. in excess of the maximum co-efficient found by experiment.

Comparing D'Arcy's and Kirkwood's formulæ for tuberculated pipe, the co-efficients of the latter are the greater for all the diameters given. As in the case of clean pipe, D'Arcy's co-efficient for tuberculated pipe increases very little for the large diameters.

TABLE 34. Giving the value of c in the formula $v = c\sqrt{rs}$ in ten different formulæ:

Diameter in ft.	Diameter in in.	VALUE OF CO-EFFICIENT c. D'Arcy, clean cast-iron pipes, formula (52)	Lampe, n = .001, formula (57)	Kutter, n = .011, s .001, formula (40)	Kutter, n = .012, s .001, formula (40)	Kutter, n = .013, s .001, formula (40)	Blackwell's formula (65)	Prony's formula (59)	Downing's formula (1)	Tuberculated: D'Arcy's formula (54)	Tuberculated: Kirkwood's formula (69)
	1	80.3	65.1	47.1			95.8	97.	100.	54.1	80.
	2	92.9	74.8	61.5			95.8	97.	100.	62.5	80.
	4	101.7	85.4	77.4			95.8	97.	100.	68.4	80.
	6	105.3	92.8	87.4	77.5	69.5	95.8	97.	100.	70.8	80.
1		109.3	106.2	105.7	94.6	85.3	95.8	97.	100.	73.5	80.
1	6	110.7	115	116.1	104.3	94.4	95.8	97.	100.	74.5	80.
2		111.5	128.5	123.6	111.3	101.1	95.8	97.	100.	74.9	80.
3		112.2	133.2	133.6	120.8	110.1	95.8	97.	100.	75.5	80.
4		112.6	139.	140.4	127.4	116.5	95.8	97.	100.	75.7	80.
5		112.8	145.2	145.4	132.3	121.1	95.8	97.	100.	75.9	80.
6		113.	150.4	149.4	136.1	124.8	95.8	97.	100.	76.	80.
7		113.1	155.	152.7	139.2	127.9	95.8	97.	100.	76.1	80.
8		113.2	159.1	155.4	141.9	130.4	95.8	97.	100.	76.1	80.
9		113.2	162.7	157.7	144.1	132.7	95.8	97.	100.	76.2	80.
10		113.3	166.1	159.7	146	134.5	95.8	97.	100.	76.2	80.
11		113.3	169.2	161.5	147.8	136.2	95.8	97.	100.	76.2	80.
12		113.3	172.1	163.	149.3	137.7	95.8	97.	100.	76.2	80
14		113.4	177.3	165.8	152	140.4	95.8	97.	100.	76.3	80.
16		113.4	182.9	168.	154.2	142.1	95.8	97.	100.	76.3	80
18		113.5	186.1	169.9	156.1	144.4	95.8	97.	100.	76.3	80
20		113.5	190.	171.6	157.7	146.	95.8	97.	100.	76.4	80.

Article 16. Values of c and $c\sqrt{r}$ for Circular Channels Flowing Full. Slopes Greater than 1 in 2640.

According to Kutter's formula, the value of c, the co-efficient of discharge, is the same for all slopes greater than 1 in 1000, that is, *within these limits, c is constant.* We further find that up to a slope of 1 in 2640 the value of c is, for all practical purposes, constant, and even up to a slope of 1 in 5000 the difference in the value of c is very little. This is well exemplified in Table 35, which is compiled from Table 19.

TABLE 35. Giving the value of c for different values of $\sqrt{r}$ and s in Kutter's formula, with $n = .013$

$$v = c\sqrt{r} \times \sqrt{s}$$

$\sqrt{r}$	SLOPES.			
	1 in 1000	1 in 2500	1 in 3333.3	1 in 5000
	c	c	c	c
.6	93.6	91.5	90.4	88.4
1.	116.5	115.2	113.2	113.2
2.	142.6	142.8	141.1	141.2

An inspection of the values of c in Tables 15 to 27, will show the slight difference in the value of c up to a slope of 1 in 5000.

In Kutter's formula the value of c is found from an equation involving the values of r, n and s, so that any change in the value of s would cause a change in the value of c, but as the influence of s on the value of c, as shown above, is not very marked in such slopes as are usually adopted for pipes, sewers and conduits, the value of the co-efficient c has been computed for one slope, that is 1 in 1000, or $s = .001$. The value of the

co-efficient for all channels, open and closed, is *practically constant* for all values of *s with a steeper slope than 1 in 1000.* For flatter slopes than 1 in 1000, up to even 2 feet per mile, or 1 in 2640, the tables give results showing a maximum error in the case of a sewer 2 feet in diameter, and $n = .015$, of less than two per cent., and in the case of a sewer 8 feet in diameter, less than one-half per cent.; therefore, for all practical purposes, the tables are sufficiently accurate.

Article 17. Construction of Tables for Circular Channels.

The plan on which these tables are constructed will be briefly stated here, and their use will be fully explained in Article 26, page 231.

The author has computed the value of c for different sizes of channels and different values of n, from his simplified form of Kutter's formula (73). By this means the complicated form of Kutter's formula (40) is reduced to the Chezy form of formula:—

$$v = c\sqrt{r} \times \sqrt{s}$$

In a similar way, the author has reduced the complicated formulæ of D'Arcy (51) and (53), to forms better adapted to computations, formulæ (52) and (54)—and by the latter formulæ, the values of c have been computed. The values of r and a being given, and the values of c computed, the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ are computed and tabulated from Table 48 to Table 69, inclusive. These tables are all that is necessary for the rapid solution of all problems relating to pipes, sewers and conduits, by the formulæ of Kutter and D'Arcy. The author was the first to use the $\sqrt{s}$ as

a *separate factor*, and its use has simplified the application of the other factors very much. We have:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and, therefore,}$$

$$Q = ac\sqrt{r} \times \sqrt{s}$$

By selecting the proper factors and using the required formula (41) to formula (50), any problem relating to pipes, sewers and conduits, can be solved rapidly.

Article 18. The Tables as a Labor Saving Machine.

In order to show the utility of these tables as a labor saving machine, and also their correctness, an instance is given of the computation of discharge from sewers.

A few years since a report was published on the sewerage of Washington, D. C., by Captain F. V. Greene, U. S. Engineers. In this report a table is given showing the discharge of circular and egg-shaped sewers with $n = .013$, computed by Kutter's formula. Table 36 given below shows about half of the table given in Captain Greene's report, and in parallel columns is also given the discharge as computed by the tables in this work. The discrepancies are caused by Captain Greene having used 41.66 instead of 41.6 on the right hand side of formula (40). It will be seen that the results by the tables in this book are practically the same as those obtained by the use of Kutter's formula (40). It is not an exaggeration to assert, that in the computation of similar tables to these in Captain Greene's report, as much work could be done in one hour by the use of the tables in this book as could be done in twelve or more hours by the use of Kutter's formula (40).

TABLE 36. Giving discharge in cubic feet per second of circular and egg-shaped sewers, based on Kutter's formula, with $n = .013$.

Dimensions of Sewers.	Discharge in Cubic Feet per Second.					
	Slope 1 in 100		Slope 1 in 200		Slope 1 in 300	
	By Kutter's formula.	By Flynn's Tables.	By Kutter's formula	By Flynn's Tables.	By Kutter's formula	By Flynn's Tables.
1′ 0″ circular.	3.39	3.35	2.40	2.37	1.96	1.93
1′ 3″ "	6.25	6.19	4.42	4.37	3.61	3.57
1′ 6″ "	10.35	10.21	7.32	7.22	5.97	5.9
1′ 9″ "	15.78	15.57	11.16	11.01	9.10	8.99
2′ 0″ "	22.68	22.46	16.04	15.88	13.08	12.97
10′ 0″ "	1673.7	1670.9	1183.3	1181.5	965.7	964.7
20′ 0″ "	10240.	10256	7240.	7252.	5909.	5921.
EGG-SHAPED.						
2′ 0″ x 3′ 0″...	36.69	36.49	25.94	25.8	21.17	21.06
2′ 6″ x 3′ 9″...	65.85	66.8	46.56	47.23	39.99	38.57
3′ 0″ x 4′ 6″...	109.84	109.2	77.66	77.21	63.38	63.04
3′ 6″ x 5′ 3″...	167.3	165.4	118.3	117.	96.5	95.5
4′ 0″ x 6′ 0″...	240.	236.6	169.7	167.4	138.5	136.8
4′ 6″ x 6′ 9″...	325.	324.	229.8	229.1	187.5	187.1
5′ 0″ x 7′ 6″...	429.2	429.1	303.5	303.4	247.7	247.7

In Table 37, with $n = .011$, the same accordance is shown by the use of Kutter's formula (40) and Flynn's tables.

TABLE 37, Giving the velocity in feet per second in pipes, sewers, conduits, by Kutter's formula, with $n = .011$.

Diameter in feet	Slope 1 in	Velocity by Kutter's formula (40)	Velocity by Flynn's Tables	Diameter in feet	Slope 1 in	Velocity by Kutter's formula (40)	Velocity by Flynn's Tables.
1	66	5.34	5.25	4	66	14.44	14.34
1	2640	.81	.83	4	2640	2.24	2.27
2	66	8.91	8.8	6	66	18.91	18.82
2	2640	1.36	1.39	6	2640	2.94	2.98

It will be seen that the results as given by the rapid method of the tables may, for all practicable purposes, be taken as identical to those given by the use of the troublesome and tedious formula (40).

Should the engineer, however, prefer to use the formula (40), even then the tables will give a ready means of checking the computations.

Article 19. Discussion on Kutter's Formula.

The following notes by the Author on Kutter's formula (40), with reference to Molesworth's Kutter, were published in the Transactions of the Technical Society of the Pacific Coast of January, 1886. They are inserted here as they contain some useful information on Kutter's formula (40).

In that admirable and useful work, "Molesworth's Pocket Book of Engineering Formulæ," (21st edition), a modified form of Kutter's formula for pipe discharge is given, in which the value of

$$c = \frac{181 + \frac{.00281}{s}}{1 + .026\left(41.6 + \frac{.00281}{s}\right)} \quad \ldots\ldots\ldots\ldots (70)$$

For facility of reference I will call this formula Molesworth's Kutter (70).

No mention is made by Molesworth of the value of n, that is, as to whether the formula is intended to apply to pipes having a *rough* or a *smooth* inner surface. An investigation will, however, show that his formula is accurately applicable to *only one diameter*, that is, to a diameter of one foot and with the value of $n = .013$.

The value of the term $\frac{n}{\sqrt{r}}$ in formula (40), is given

by Molesworth in formula (70), as a *constant* quantity, and $=.026$, whereas, in fact, it is a variable quantity, its value—with the same value of n—changing with every change in the hydraulic mean radius or diameter of pipe.

Now, assuming the value of n taken by Molesworth to be $=.013$ and substituting this value for n in Kutter's formula (40), we have:—

$$c=\frac{41.6+\frac{1.811}{.013}+\frac{.00281}{s}}{1+\left(41.6+\frac{.00281}{s}\right)\frac{.013}{\sqrt{r}}}$$

$$\therefore c=\frac{181+\frac{.00281}{s}}{1+\left(41.6+\frac{.00281}{s}\right)\frac{.013}{\sqrt{r}}}\cdots\cdots\cdots\cdots(71)$$

but by Molesworth's Kutter (70)

$$\frac{.013}{\sqrt{r}}=.026$$

$$\therefore\ \sqrt{r}=.5$$

$r=.25$, and as the hydraulic mean depth of a pipe is one-fourth of the diameter,

$$\therefore\quad d=1$$

If we substitute in formula (71) for $\sqrt{r}$ its value 0.5, we have:—

$$c=\frac{181+\frac{.00281}{s}}{1+.026\left(41.6+\frac{.00281}{s}\right)}$$

which is Molesworth's Kutter (70).

It is therefore apparent that, no matter what the value of n may be, Molesworth's Kutter (70), does not give

the same results as Kutter's formula (40), as it gives a constant co-efficient of velocity, c, for all diameters having the same slope and the same value of n.

Kutter's formula (40), has certain peculiarities which are wanting in Molesworth's Kutter, and an investigation will show that Molesworth's Kutter differs materially from Kutter's formula (40), and that its application, except to one diameter, is sure to lead to serious error. I will briefly explain:

1. By Kutter's formula (40), the value of c, or the volocity, changes with every change in the value of r, s, or n, and with the *same slope* and the same value of n, the value of c increases with the increase of r, that is, with the increase in diameter. It is on this variability of its co-efficient to suit the different changes of slope, diameter and lining of channel, that the accuracy of Kutter's formula depends. By Molesworth's Kutter a change in the diameter, other things remaining the same, does not affect the value of c. With the same slope the value of c is constant for all diameters.

As an instance, with a slope of 1 in 1000:—

FORMULÆ.	6 inches diameter. $c =$	20 feet diameter. $c =$
By Kutter's formula (40)	69.5	146.
Molesworth's Kutter (70).........	85.3	85.3

It will thus be seen that the value of c by Kutter's formula (40), when $s = .001$, has a large range, from 69.5 to 146.0, showing an increase of 111 per cent. from a diameter of 6 inches to a diameter of 20 feet.

It will be further found that Molesworth's formula gives the value of c, and therefore the value of the velocity and discharge, too high for diameters less than

one foot, and too low for diameters above one foot, and the more the diameter differs from one foot the greater is the error. In these respects it follows the errors of the old formulæ.

2. According to Kutter's formula (40) the value of c increases with the increase of slope for all diameters whose hydraulic mean depth is less than 3.281 feet—one metre—and with a hydraulic mean depth greater than 3.281 feet, an increase of slope gives a diminution in the value of c.

The small table, herewith given, shows this:—

Table 38. Giving the co-efficients of discharge, c, in circular pipes of different diameters and different grades with $n = .013$.

Formulæ.	12 feet diameter.		20 feet diameter.	
	1 in 1000.	1 in 40.	1 in 1000.	1 in 40.
Molesworth's Kutter $c =$...	85.3	86.9	85.3	86.9
Kutter's formula $c =$......	137.7	137.9	146.	145.7

It will thus be seen that by Kutter's formula (40), when $r = 3$ feet, that is, less than 3.281 feet, an increase in the slope from 1 to 1000 to 1 in 40, causes a slight increase in the co-efficient, but when r is 5 feet, that is, more than 3.281 feet, the same increase in the slope causes a slight diminution in the value of c.

By Molesworth's Kutter formula (70), when $r = 3$ feet, an increase in the slope from 1 in 1000 to 1 in 40 causes a greater proportional increase in the co-efficient than Kutter gives, and when $r = 5$ feet the value of the co-efficient does not diminish with the increase of slope, but, on the contrary, it increases with the increase in slope, and its value is the same as when $r = 3$ feet.

3. By Kutter's formula (40), when the hydraulic mean depth is equal to 3.281 feet, one metre, the value of c is *constant* for all slopes, and is $= \frac{1.811}{n}$, which in this case $= \frac{1.811}{.013} = 139.31$.

Let $r = 3.281$ feet, and, therefore, $\sqrt{r} = \sqrt{3.281} = 1.811$, substitute this value in Kutter's formula (40), and we have

$$c = \frac{41.6 + \frac{1.811}{n} + \frac{.00281}{s}}{1 + \left(41.6 + \frac{.00281}{s}\right)\frac{n}{1.811}}$$

and $\therefore c = \frac{1.811}{n}$, and when $n = .013$, $c = 139.31$.

This is the only instance, I believe, where Kutter's formula (40) gives a constant co-efficient with a change of slope. By Molesworth's Kutter (70), on the contrary, the value of c changes with every change of slope when $r = 3.281$.

It is evident that Molesworth's Kutter was adopted in order to simplify the application of Kutter's formula (40), but its simplification is of no practical use, as it gives very inaccurate results.

As shown above, with the exception of its application to one diameter, the formula is not Kutter's, although in appearance bearing a resemblance to it.

However, a modification of Kutter's formula can be made simpler in form than even Molesworth's Kutter (70), and giving results near enough for all practical purposes to those obtained by the use of the more complicated Kutter formula (40).

The value of c in Kutter's formula (40), with a slope of 1 in 1000, and $n = .013$ is thus expressed:—

$$c = \frac{41.6 - \frac{1.811}{.013} + \frac{.00281}{.001}}{1 - \left(41.6 - \frac{.00281}{.001}\right)\frac{.013}{\sqrt{r}}}$$

$$\therefore c = \frac{183.72}{1 + \left(44.41 \times \frac{.013}{\sqrt{r}}\right)} \quad \text{........} (72)$$

The following table will show the value of the co-efficient c for several slopes and diameters according to formulæ (70), (40) and (72).

TABLE 39. Giving values of c, the co-efficient of discharge, according to different modifications of Kutter's formula with $n = .013$.

	Moles-worth's Kutter (70) $c =$	Kutter's formula (40) $c =$	Flynn's Kutter (72) $c =$
6 inch diameter, slope 1 in 40....	86.9	71.5	69.5
6 inch diameter, slope 1 in 1000..	85.3	69.5	69.5
4 feet diameter, slope 1 in 400 ...	87.2	117.	116.5
4 feet diameter, slope 1 in 1000...	85.3	116.5	116.5
8 feet diameter, slope 1 in 700 ...	85.8	130.5	130.5
8 feet diameter, slope 1 in 2600...	82.9	129.8	130.5

This table shows the close agreement of formula (72) with Kutter's formula (40), and it also shows the inaccurate results obtained by the use of Molesworth's Kutter.

The first column of this table shows that a formula with a *constant* value of $c = 85$, that is:—

$$v = 85\sqrt{rs}$$

will give results differing in an extreme case only $2\frac{1}{2}$ per cent. from Molesworth's Kutter, and in the greater number of cases differing only about one per cent.

The second column of the table shows the wide range of the co-efficient c by Kutter's formula (40) from 69.5 to 130.5, to suit the different changes in the hydraulic mean depth and slope.

The objection to the old formulæ was that they gave velocities too high for small pipes and channels, and too low for large pipes and channels. The following table will show that the same inaccurate results are obtained by the use of Molesworth's Kutter (70).

TABLE 40. Giving the mean velocity, in feet per second, of pipes of different diameters and grades, with $n = .013$.

	Velocity in Feet per Second.		
	Molesworth (70).	Kutter (40).	Flynn's Kutter (72).
6 inches diameter, slope 1 in 40..	4.86	4.	3.89
6 inches diameter, slope 1 in 1000	.95	.78	.78
4 feet diameter, slope 1 in 400...	4.36	5.85	5.83
4 feet diameter, slope 1 in 1000..	2.70	3.68	3.68
8 feet diameter, slope 1 in 700...	4.59	6.97	6.97
8 feet diameter, slope 1 in 2600..	2.30	3.60	3.62

This table shows that there is a wide difference between the velocities obtained by Molesworth's Kutter (70) and Kutter's formula (40), and it further shows that for the slopes usually adopted in practice for pipes, sewers, conduits, etc., that is, for slopes not flatter than 2 feet per mile, or 1 in 2640, formula (72) will give velocities that, for all practical purposes, may be consid-

ered as almost identical with the velocities obtained by Kutter's formula (40).

In Van Nostrand's Engineering Magazine for September, 1886, is a letter on this subject from Mr. Guildford Molesworth, the author of the Pocket Book, of which the following is a copy:

To the Editor of Van Nostrand's Magazine:

Mr. Flynn's criticism of my modification of Kutter's formula for pipes has just reached me. Mr. Flynn is quite correct. The formula as it stands in page 25 of the twenty-first edition of my pocket book has an omission of $\sqrt{d}$. As I originally framed it, it stood thus:

$$c = \frac{181 + \frac{.00281}{s}}{1 + \frac{.026}{\sqrt{d}}\left(41.6 + \frac{.00281}{s}\right)}$$

Unfortunately, the omission of $\sqrt{d}$ escaped my observation in correcting the proofs of this twenty-first edition.

Taking the side cases which Mr. Flynn has worked out, a comparison of Kutter's formula and my modification of it for pipes, as corrected, stands thus:

Diameter of Pipe.	Slope 1 in	Kutter.	Molesworth.
6 inches	40	71.50	71.48
6 inches	1000	69.50	69.79
4 feet	400	117.	117.
4 feet	1000	116.5	116.55
8 feet	700	130.5	130.68
8 feet	2600	129.8	129.93

The two formulæ are thus far substantially identical in results, though differing slightly in form. GUILDFORD MOLESWORTH.

Simla, India, May 17, 1886.

Article 20. Flynn's Modification of Kutter's Formula.

The author has reduced Kutter's formula for slopes up to 1 in 2640, into the simplified form given in formula (73).

Referring to the simplified form of Kutter's formula

(72), if we call the numerator on the right hand side of the equation K, for any value of n we have:—

$$c = \frac{K}{1 + \left(44.41 \times \frac{n}{\sqrt{r}}\right)}$$

$$\text{and } v = \left\{ \frac{K}{1 + \left(44.41 \times \frac{n}{\sqrt{r}}\right)} \right\} \sqrt{rs} \ \ldots\ldots\ldots\ldots (73)$$

In the following table the value of K is given for the several values of n.

Table 41. Giving the value of K for use in Flynn's modification of Kutter's formula:

n	K	n	K	n	K	n	K	n	K
.009	245.63	.012	195.33	.015	165.14	.018	145.03	.021	130.65
.010	225.51	.013	183.72	.016	157.6	.019	139.73	.022	126.73
.011	209.05	.014	137.77	.017	150.94	.020	134.96	.0225	124.9

To further simplify formula (73), the value of $\sqrt{r}$ for a large range of diameters will be found in Table (42).

If, therefore, in the application of formula (73), within the limits of n as given in the table, we substitute for n, K, and $\sqrt{r}$, their values, we have a simplified form of Kutter's formula (40).

For instance, when $n = .011$, and $d = 3$ feet, we have:—

$$v = \left\{ \frac{209.05}{1 + \left(44.41 \times \frac{.011}{.866}\right)} \right\} \times \sqrt{rs}$$

TABLE 42. Giving values of $\sqrt{r}$ for circular pipes, sewers and conduits of different diameters:—

Diamet'r		$\sqrt{r}$	Diamet'r		$\sqrt{r}$	Diamet'r		$\sqrt{r}$	Diamet'r		$\sqrt{r}$
Ft.	Ins.	in Feet	Ft.	Ins.	in Feet.	Ft.	Ins.	in Feet	Ft.	Ins.	in Feet
	5	.323	2	9	.829	5	1	1.127	10		1.581
	6	.354	2	10	.842	5	2	1.137	10	3	1.601
	7	.382	2	11	.854	5	3	1.146	10	6	1.620
	8	.408	3		.866	5	4	1.155	10	9	1.639
	9	.433	3	1	.878	5	5	1.164	11		1.658
	10	.456	3	2	.890	5	6	1.173	11	3	1.677
	11	.479	3	3	.901	5	7	1.181	11	6	1.696
1		.500	3	4	.913	5	8	1.190	11	9	1.714
1	1	.520	3	5	.924	5	9	1.199	12		1.732
1	2	.540	3	6	.935	5	10	1.208	12	3	1.750
1	3	.559	3	7	.946	5	11	1.216	12	6	1.768
1	4	.577	3	8	.957	6		1.225	12	9	1.785
1	5	.595	3	9	.968	6	3	1.250	13		1.803
1	6	.612	3	10	.979	6	6	1.275	13	3	1.820
1	7	.629	3	11	.990	6	9	1.299	13	6	1.837
1	8	.646	4		1.	7		1.323	13	9	1.854
1	9	.661	4	1	1.010	7	3	1.346	14		1.871
1	10	.677	4	2	1.021	7	6	1.369	14	6	1.904
1	11	.692	4	3	1.031	7	9	1.392	15		1.936
2		.707	4	4	1.041	8		1.414	15	6	1.968
2	1	.722	4	5	1.051	8	3	1.436	16		2.
2	2	.736	4	6	1.061	8	6	1.458	16	6	2.031
2	3	.750	4	7	1.070	8	9	1.479	17		2.061
2	4	.764	4	8	1.080	9		1.500	17	6	2.091
2	5	.777	4	9	1.089	9	3	1.521	18		2.121
2	6	.790	4	10	1.099	9	6	1.541	19		2.180
2	7	.804	4	11	1.109	9	9	1.561	20		2.236
2	8	.817	5		1.118						

Article 21. D'Arcy's Formulæ.

M. H. D'Arcy's experiments on the flow of water in new and old cast-iron pipes are the most thorough and elaborate investigations of the kind which have ever been carried out. He demonstrated that the degree of roughness of the wetted surface has an important effect on the discharge of the pipe.

M. D'Arcy had observed, in the course of his experience on waterworks, that in proportion to the smoothness of the inner surface of the pipe, so was its

discharge increased. He had at his disposal ample means to carry out experiments to prove this. He was an engineer eminently fitted to carry out such experiments, on account of his great scientific attainments, and his practical experience gained whilst in charge of City Waterworks, and the results of his observations fully justified the confidence placed in his ability.

It is to be regretted that his experiments did not extend to large pipes. He made experiments with 22 pipes of cast and wrought iron, sheet iron covered with bitumen, and lead and glass, but none of them were of large dimensions. His experiments on pipes fully justified his former experience, and Bazin's observations on small open channels gave further testimony to the same effect.

The experiments of D'Arcy and Bazin * were afterwards of great value to Kutter in his hydraulic investigations.

After the publication of the results of D'Arcy's observations in the French, Mr. J. B. Francis, M. Am. Soc. C. E.† presented his formulæ in a form suitable to feet measures.

Mr. J. W. Adams, M. Am. Soc. C. E., in *Engineering News* of March 10th, 1883, writes:—

"When the Loch Katrine Water Works for Glasgow were being extended some years since, a portion of the distance was carried over low grounds by a cast-iron trough 6½ feet deep and 8 feet in width, supported on masonry piers, and giving good opportunity to determine the daily flow. By this and other means it was found that the cast-iron pipes, 4 feet in diameter, which with a fall of 1 in 1056 on the rest of the line, had been computed to carry 21,000,000 gallons, were really dis-

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* Recherches Hydrauliques.

† Transactions American Society of Civil Engineers. Vol. II.

charging daily 23,430,000 gallons. The engineer, Mr. Gale, brought the matter to Professor Rankine's attention; who, in a paper and subsequent discussion before the Institution of Engineers of Scotland, March 17th, 1869, uses this language: 'It might be interesting to the Institution to know that there was a formula which agreed exactly with the results of Mr. Gale's experiments. Suppose that before these four-feet pipes were laid, the probable discharge had been calculated by D'Arcy's formula, the result would have differed *by one part in a thousand*, from the actual discharge, which was 23,430,000 gallons daily. This went to show that they now possessed a general formula for the flow of water in pipes, and the resistance to that flow, which applied to large as well as small pipes (it applied to pipes of an inch in diameter), and from Mr. Gale's experiments they would see that it also applied to pipes four *feet* in diameter.' The Glasgow pipes had been coated with Dr. Smith's process, and were treated as clean pipes and calculated by the formula (for clean pipes). I think that D'Arcy's experiments conducted as they were under circumstances which contributed in every way to inspire confidence. Mr. Francis' labors in presenting this formula to us in English dress, with the prestige growing out of his well-known capacity for careful investigation and computation, and Professor Rankine's indorsement of its applicability to all conditions of pipe discharge up to four feet diameter, must be considered as establishing the practical value of *this* special formula for the flow through *iron pipes*."

Mr. W. Humber, C. E., in his work on "Water Supply," states:—

"That which is known as D'Arcy's formula, in pipes of large diameter, appears to approach in its results nearer to the actual discharge than any other, and it was

the opinion of Professor Rankine, that the resistance decreases to a greater extent in pipes of larger diameter than has been previously supposed. The experiments were made with, and the formula of D'Arcy deduced from, pipes which had been long in use without offering any impediment from incrustation."

Example 23 is an illustration of the accuracy of D'Arcy's formula, where the actual discharge from a 48-inch pipe was found to be the same as that given by computing by D'Arcy's formula.

It was found, however, that after some time the discharge gradually fell off, and, though in the first instance, the amount was 50 per cent. larger than that given by the old formula, still it gradually diminished, though the pipes still continued to discharge more than the amount gained by the old formula. The degree of roughness of the pipe was a measure of its discharging capacity.

In a paper presented to the Technical Society of the Pacific Coast, on February 6, 1885, the author simplified D'Arcy's formula (51), into the form of formula (52):—

$$v = \left(\frac{155256\, d}{12\, d + 1} \right)^{\frac{1}{2}}$$

This was done in order to obtain a formula adapted to the preparation of a table facilitating the use of D'Arcy's formula. In a similar way the author has simplified D'Arcy's formula (53), for old cast-iron pipe lined with deposit, into the form given in formula (54).

Table 48 is for clean cast-iron pipe, and table 49, for old cast-iron pipe lined with deposit.

D'Arcy's formula for finding the mean velocity in clean cast-iron pipes.

For feet measures D'Arcy's formula for mean velocity in clean cast-iron pipes is:—

$$v = \left\{ \frac{rs}{.00007726 + \dfrac{.00000162}{r}} \right\}^{\frac{1}{2}}$$

and from this we have:—

$$s = \left(.00007726 + \frac{.00000162}{r} \right) \frac{v^2}{r}$$

In order to simplify, *substitute for r in feet the diameter d in inches*, and we have

$$s = \left(.00007726 + \frac{.00000162 \times 48}{d} \right) \frac{48\ v^2}{d}$$

$$\therefore\ s = \left(.00370848\ d + .00373248 \right) \frac{v^2}{d^2}$$

As the change will not materially affect the result, Mr. J. B. Francis, C. E., simplifies this into the form

$$s = .00371 \left(d + 1 \right) \frac{v^2}{d^2} \ldots\ldots\ldots\ldots \text{(A)}$$

$$\therefore\ v = \left(\frac{s\,d^2}{.00371\ (d + 1)} \right)^{\frac{1}{2}}$$

In order, however, to further simplify the equation into the Chezy form of formula, which is the form required for the preparation and use of the tables adopted by the writer, and given in this book, let equation (A) be transformed into one with the *diameter d in feet*, and it becomes:—

$$s = .00371 \left(12\ d + 1 \right) \frac{v^2}{144\ d^2}$$

Therefore, for clean iron pipes

$$v = \left\{ \frac{144\ d^2\ s}{.00371\ (12\,d + 1)} \right\}^{\frac{1}{2}}$$

but $d^2 = 16\,r^2 = 16\,r \times r = 4\,d \times r$ substitute this value for d^2 in the last equation, and

$$v = \left\{ \frac{144 \times 4\,d \times r \times s}{.00371\ (12\,d + 1)} \right\}^{\frac{1}{2}}$$

Therefore, *for feet measures*, D'Arcy's formula for velocity is simplified into

$$v = \left(\frac{155256\ d}{12\,d + 1}\right)^{\frac{1}{2}} \times \sqrt{rs}$$

and putting the first factor on the right-hand side of the equation $= c$, we have

$$v = c\sqrt{rs} - c\sqrt{r} \times \sqrt{s}$$

D'Arcy's formula for finding the mean velocity in old cast-iron pipes.

Mr. J. B. Francis, M. Am. Soc. C. E., has given D'Arcy's formula for the Flow of Water through old cast-iron pipes lined with deposit as:—

$$s = .0082\left(d + 1\right)\frac{v^2}{d^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(B)}$$

where s and v have the same values as given at pages 6 and 7, and $d =$ diameter in inches.

In order, however, to further simplify the equation into the Chezy form of formula, which is the form required for the preparation and use of the tables, as already stated, let formula (B), be transformed into one with the *diameter d in feet*, and it becomes:—

$$s = .0082\left(12\,d + 1\right)\frac{v^2}{144\,d^2}$$

$$\therefore v = \left(\frac{144\,d^2\,s}{.0082\,(12\,d + 1)}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(C)}$$

but $d = 4\,r$, and $d^2 = d \times 4\,r$, substitute these values in formula (C) for d_2, and:—

$$v = \left(\frac{144\,d \times 4rs}{.0082\,(12\,d + 1)}\right)^{\frac{1}{2}}$$

and therefore, for feet measures D'Arcy's formula for

the mean velocity in old cast-iron pipes lined with deposit is simplified into the form:—

$$v = \left(\frac{70243.9\,d}{12\,d + 1}\right)^{\frac{1}{2}} \times \sqrt{rs}$$

and putting the first factor in parenthesis on the right hand side of the equation $= c$, we have:—

$$v = c\sqrt{rs}$$

Article 22. Comparison of the Co-efficients for Small Diameters of the Formulæ of D'Arcy, Kutter, Jackson and Fanning.

$$v = c\sqrt{r} \times \sqrt{s}$$

In tables 48 to 57 inclusive, the values of the factors of Kutter's formula are not given for diameters less than 5 inches. Mr. L. D'A. Jackson, C. E., in his *Hydraulic Manual*, states:—

"For the present, and until further experiments have thrown more light on the subject, it may be assumed that the co-efficient of discharge for all full cylindrical pipes, having a diameter less than 0.4 feet, will be the same as those of that diameter."

Although Mr. Jackson's opinion is entitled to great weight, still the facts all tend to prove that the co-efficients of diameters below 5 inches should diminish with the diminution of diameter. The smaller the diameter the more effect will the roughness of the surface have in diminishing the discharge. Table 48 shows that Kutter's co-efficient for 5 inches diameter with $n = .011$ is 82.9, and therefore, according to Mr. Jackson, all the diameters from 5 inches to ¼ inch should have a co-efficient of 82.9. This is contrary to the principle of Kutter's formula, the accuracy of which is due to the fact that, other things being equal, its co-

efficients vary with the diameter. The following proofs are given in support of the opinion that co-efficients of diameters below 5 inches should diminish according to the diminution of diameter.

Table 43. Of co-efficients (c) from the formulæ of D'Arcy, Kutter, Jackson and Fanning, for small pipes below 5 inches in diameter,

$$v = c\sqrt{rs}$$

Diameter in inches.	(c) D'Arcy's co-efficient for clean pipes.	(c) Kutter's co-efficient from formula $n = .011$ $s = .001$	(c) Kutter's co-efficient recommended by L. D'A. Jackson.	(c) Fanning's co-efficient for clean iron pipes.
$\frac{3}{8}$	59.4	32.	82.9	
$\frac{1}{2}$	65.7	36.1	82.9	
$\frac{3}{4}$	74.5	42.6	82.9	
1	80.4	47.4	82.9	80.4
$1\frac{1}{4}$	84.8	51.9	82.9	
$1\frac{1}{2}$	88.1	55.4	82.9	88.
$1\frac{3}{4}$	90.7	58.8	82.9	92.5
2	92.9	61.5	82.9	94.8
$2\frac{1}{2}$	96.1	66.	82.9	
3	98.5	70.1	82.9	96.6
4	101.7	77.4	82.9	103.4
5	103.8	82.9	82.9	

1. In Table 43 the co-efficients of Darcy's formula are seen to diminish with the diminution of diameter. At 5 inches diameter the co-efficient is 103.8, and at $\frac{3}{8}$ inch diameter 59.4.

2. In Table 43 the co-efficients of Fanning's formula diminish from 4 inches diameter with a co-efficient of 103.4, to 1 inch diameter with a co-efficient of 80.4.

These co-efficients are derived from the mean velocities in clean pipes with a slope of 1 in 125 given in Fanning's tables.

3. In Table 43 the co-efficients, as found by Kutter's formula with a slope of 1 in 1000, and $n = .011$, are for 5 inches diameter, 82.9, and for $\frac{3}{8}$ inch diameter, 32.0.

The facts, therefore, show that the co-efficients diminish from a diameter of 5 inches to smaller diameters, and it is a safer plan to adopt co-efficients varying with the diameter than a constant co-efficient. No opinion is advanced as to what co-efficients should be used with Kutter's formula for small diameters. The facts are simply stated, giving the results of well-known authors.

As the co-efficients of D'Arcy's formula vary only with the diameter, the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ given in Tables 48 and 49 for D'Arcy's formula, are practically the exact values for all diameters and slopes given, and the results found by the use of the tables will be the same as the results found by using the formula.

In Tables 50 to 67, the values of $c\sqrt{r}$ and $ac\sqrt{r}$ for Kutter's formula sometimes differ, when the slope is flatter than 1 in 1000, by a small quantity from the actual values as found by the use of formula (40). These values by Kutter's formula depend not only on r, but also on n and s, so that a change in any of these three quantities causes a change in the values of $c\sqrt{r}$ and $ac\sqrt{r}$. It is found, however, that the slope of 1 in 1000 will give co-efficients which practically differ very little from the co-efficients derived from the slopes usually given to lines of pipes, sewers and conduits.

The values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$, from Kutter's formula given in the tables 50 to 67, have been computed for a slope of 1 in 1000, and they give values of $c\sqrt{r}$ and $ac\sqrt{r}$ near enough for practical work.

Article 23. Pipes, Sewers, Conduits, etc., Having the Same Velocity.

The columns $c\sqrt{r}$ in Tables 48 to 57, inclusive, for circular channels, and Tables 59 to 67, inclusive, for egg-shaped sewers, can be used to compare velocities, as, other things being equal, the velocities are proportional to $c\sqrt{r}$. The formula:—

$v = c\sqrt{r} \times \sqrt{s}$, is proof of this.

For example, a circular pipe or sewer 4 feet in diameter flowing full, with a value of $n = .013$, and a slope of 1 in 1500, has a mean velocity of 2.988 feet, that is, practically, 3 feet per second. In Table 54 we find that this channel has $c\sqrt{r} = 116.5$. Now all pipes, under different values of n, of different diameters, having the same grade and the same value of $c\sqrt{r}$, will have the same velocity.

Again, the slope being equal, we can find, merely by inspection, the dimensions of an egg-shaped sewer, of a different value of n, flowing full, two-thirds full, or one-third full, that will have the same velocity as a circular sewer with a different value of n flowing full.

Thus, taking the circular sewer mentioned above of 4 feet in diameter and $n = .013$, and we want to find the dimensions of an egg-shaped sewer flowing two-thirds full, that, with $n = .015$, and the same grade, will have the same velocity. In Table 66 of egg-shaped sewers flowing two-thirds full and with $n = .015$, we find opposite a sewer having the dimensions of 4′x6′, that $c\sqrt{r} = 116.5$, therefore a circular sewer 4 feet in diameter with $n = .013$, will, with the same slope, have the same velocity as an egg-shaped sewer 4′x6′ with $n = .015$, and flowing two-thirds full.

Table 47, giving the values of the hydraulic mean

depth, r, of circular pipes, etc., and Table 58, giving the values of r for egg-shaped sewers, can be used with great advantage in a variety of problems in comparing the velocities in pipes, sewers and conduits.

In the following table, given to illustrate what has been just stated, the nearest values of $c\sqrt{r}$ given in the working tables are inserted:—

TABLE 44. Circular Pipes, Sewers and Conduits having the same mean velocity and the same grade, but with different diameters and different values of n, based on Kutter's formula:—

No. of Table	Value of n	Diameter, Ft.	Ins.	$c\sqrt{r}$	Slope 1 in 1500 $\sqrt{s}$	Velocity in feet per second.	Remarks.
50	.009	2	2	117.	.02582	3.021	Circular.
51	.01	2	7	116.8	.02582	3.016	Circular.
52	.011	3	1	117.9	.02582	3.044	Circular.
53	.012	3	6	116.3	.02582	3.003	Circular.
54	.013	4		116.5	.02582	2.988	Circular.
55	.015	5	1	117.1	.02582	3.023	Circular.
56	.017	6	3	117.6	.02582	3.036	Circular.
57	.020	8		117.2	.02582	3.026	Circular.

The mean velocity of egg-shaped sewers can be compared in the same way, or can be compared with circular sewers. Thus, let us find the dimensions of egg-shaped sewers having the same velocity and the same grade as the circular sewers in Table 44, but with different values of n.

TABLE 45. Egg-shaped sewers having the same velocity and the same grade, but with different dimensions and different values of n:—

No. of Table	Value of n	Dimensions	$c\sqrt{r}$	Slope 1 in 1500 $\sqrt{s}$	Velocity in feet per second.	Remarks.
59	.011	2′ 8″ x 4′ 0″	118.	.02582	3.047	Full depth.
60	.011	2′ 6″ x 3′ 9″	119.9	.02582	3.096	$\frac{2}{3}$ full depth.
61	.011	3′ 8″ x 5′ 6″	116.4	.02582	3.005	$\frac{1}{3}$ full depth.
62	.013	3′ 6″ x 5′ 3″	117.6	.02582	3.036	Full depth.
63	.013	3′ 2″ x 4′ 9″	116.5	.02582	3.008	$\frac{2}{3}$ full depth.
64	.013	4′ 10″ x 7′ 3″	116.5	.02582	3.008	$\frac{1}{3}$ full depth.
65	.015	4′ 4″ x 6′ 6″	116.	.02582	2.995	Full depth.
66	.015	4′ 0″ x 6′ 0″	116.5	.02582	3.008	$\frac{2}{3}$ full depth.
67	.015	6′ 2″ x 9′ 3″	117.3	.02582	3.028	$\frac{1}{3}$ full depth.

Article 24. Pipes, Sewers and Conduits Having the Same Discharge.

By an exactly similar method to that adopted for velocities in Article 23, we can use the columns of $ac\sqrt{r}$ for finding equivalent discharging pipes, sewers and conduits. We can also find the dimensions of a single sewer having a discharge equivalent to that of several other sewers. For example, three circular sewers have, at different times, been constructed to an outfall on a river. The sewers are, respectively, 10, 12 and 18 inches in diameter. The grade is 1 in 300, and their value of $n = .013$. What must be the dimensions of an egg-shaped sewer that, flowing two-thirds full depth, with the same value of n and the same grade, will have a discharge double that of the three circular sewers mentioned?

In Table 54, of circular sewers with $n = .013$, we find a

$$10 \text{ inch sewer has } ac\sqrt{r} = 20.095$$
$$12 \text{ inch sewer has } ac\sqrt{r} = 33.497$$
$$18 \text{ inch sewer has } ac\sqrt{r} = 102.140$$

Therefore, the three circular sewers $ac\sqrt{r} = 155.732$

Now $155.732 \times 2 = 311.464$, which is the value of $ac\sqrt{r}$ of the water section of the new sewer.

In Table 63 of egg-shaped sewers flowing two-thirds full depth with $n = .013$, we find opposite a sewer $2' \ 2'' \times 3' \ 3''$ that $ac\sqrt{r} = 317.19$, therefore the required sewer is $2' \ 2'' \times 3' \ 3''$.

In order to further illustrate this subject, Table 46 is given. This table further shows the effect of the value of n; for a pipe 2 feet 2 inches diameter with a value of $n = .009$, has practically the same discharge as a 2 foot 9 inch pipe with a value of $n = .015$.

TABLE 46. Pipes, Sewers and Conduits, having the same grade and the same or nearly the same discharge, but with different diameters and different values of n.

No. of Table.	Value of n.	DIAMETER.		$ac\sqrt{r}$	Slope 1 in 1500 $\sqrt{s}$	Discharge in cubic ft. per second	Remarks.
		Feet.	Inches.				
50	.009	2	2	431.5	.02582	11.14	Circular.
51	.01	2	3	421.9	.02582	10.89	Circular.
52	.011	2	5	457.1	.02582	11.8	Circular.
53	.012	2	6	452.1	.02582	11.67	Circular.
54	.013	2	7	450.5	.02582	11.63	Circular.
55	.015	2	9	451.2	.02582	11.65	Circular.

In the same manner the discharge of egg-shaped sewers can be compared.

The discharge is not exactly the same for each pipe, for the reason that the exact value of $ac\sqrt{r} = 431.5$ could not be found opposite the diameters in tables, and, therefore, the nearest value to 431.5 was taken.

What has been shown in this and the foregoing articles is sufficient to demonstrate to the practical engineer the rapidity with which problems relating to pipes, sewers and conduits can be solved by the tables in this work.

Article 25. Egg-Shaped Sewers.

Where the volume of sewage fluctuates, the oval form of sewer is the best adapted with a small discharge, to give a velocity sufficient to prevent the deposit of silt, as its hydraulic mean depth is greater for small volumes of flow than the circular sewer.

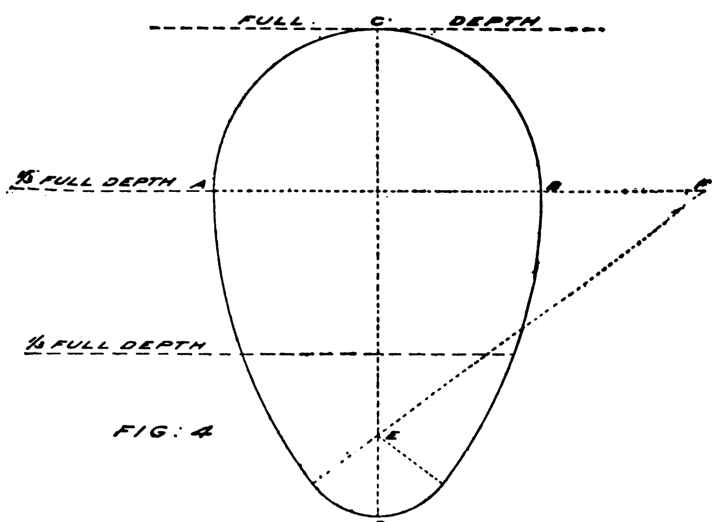

Fig. 4. Egg-shaped Sewer.

The egg-shaped sewer treated of in this work has its depth, or vertical diameter, equal to 1.5 times its greatest

transverse diameter, that is, the diameter of top or arch. This form of cross-section of sewer is illustrated in Figure 4.

$D = AB$ = greatest transverse diameter, that is, the diameter of top or arch $= \frac{2H}{3}$.

$H = CD$ = depth of sewer or vertical diameter $= 1.5D$.

$B = ED$ = radius of bottom or invert $= \frac{H}{6}$.

$R = AF$ = radius of sides $= H$.

By reference to Table 69, it will be seen that the value of the velocity of an egg-shaped sewer flowing two-thirds full depth, is always *greater* than that of the mean velocity of the same sewer flowing full depth. The discharge, however, is always greater in the sewer flowing full depth.

Article 26. Explanation and Use of the Tables.

Pipes, Sewers and Conduits.

EXAMPLE 21. *Given the diameter, length, fall and value of n of a pipe, to find its mean velocity and discharge.*

An inverted syphon, B, C, D, E, F, measured *along the line of pipe*, is five miles in length, and its outlet at F is 40 feet below the surface of the reservoir at A. The

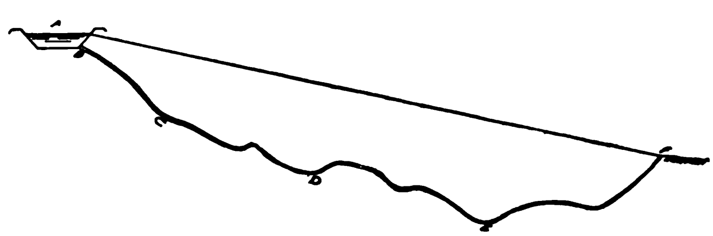

Fig. 5. Inverted Syphon.

pipe is 2 feet in diameter. It is made of sheet-iron, double riveted, dipped in hot asphaltum. This dip

gives a very smooth surface at first, but to allow for the deterioration of that surface the value of n is taken $=.013$. What is its mean velocity in feet per second, and the discharge in cubic feet per second? It is well to remember that at no part of its length should the pipe rise above the *hydraulic grade* line $A\ F$.

A fall of 40 feet in five miles is equivalent to 8 feet per mile, or 1 in 660. In Table 33, opposite a slope of 1 in 660, we find $\sqrt{s} = .038925$.

In Table 54, for circular pipes with $n = .013$, we find $a = 3.142$, $c\sqrt{r} = 71.49$ and $ac\sqrt{r} = 224.63$. Now, to find mean velocity, substitute the values of $c\sqrt{r}$ and $\sqrt{s}$ in formula (41), and we have:—

$$v = 71.49 \times .038925 = 2.783 \text{ feet per second.}$$

Again, to find the discharge, substitute the values of $ac\sqrt{r}$ and $\sqrt{s}$ in formula (45), and we have:—

$Q = 224.63 \times .038925 = 8.744$ cubic feet per second.

As a check on the above we have by formula (45):—

$Q = av$ substitute the values of a and v above found and we have:—

$Q = 3.142 \times 2.783 = 8.744$ cubic feet per second, which is the same as the discharge before found.

EXAMPLE 22. *Given the discharge and cross-sectional dimensions of a rectangular, masonry, Inverted Syphon, to find its grade or fall from surface of water at inlet to its outlet.*

At page 177 of *Irrigation Canals and other Irrigation Works*, there is a description of an inverted syphon under the Agra Canal, India. The syphon is capable of discharging 2,000 cubic feet per second. It has seven culverts each 6 feet wide and 4 feet deep. The syphon is provided with a floor of massive rough ashlar, the entrance and egress for the torrent being also built of

large stone. The culverts are covered with large stones bolted down to the piers. The length of the syphon is assumed at 200 feet. From the description given of the surface of the syphon exposed to the flow of water, we may assume its value of $n = .017$ (see page 19).

The total discharge being 2,000 cubic feet per second, therefore, each of the seven syphons has to discharge 286 cubic feet per second. The area of one culvert $= 6' \times 4' = 24$ square feet.

$$v = \frac{Q}{a} = \frac{286}{24} = 12 \text{ feet per second nearly.}$$

$$r = \frac{a}{p} = \frac{24}{20} = 1.2 \therefore \sqrt{r} = \sqrt{1.2} = 1.1 \text{ nearly.}$$

Under a slope of 1 in 1000 and opposite $\sqrt{r} = 1.1$ in Table 21, page 132, we find $c\sqrt{r} = 98.4$.

Now substitute this value of $c\sqrt{r}$ and also the value of v in formula (43), and we have:—

$$\sqrt{s} = \frac{12.}{98.4} = .121951$$

The nearest value to this, in Table 33, is .122169 opposite a slope of 1 in 67, but as the total length is 200 feet $\therefore \frac{200}{67} = 3$ feet nearly, being the head required to generate a velocity of 12 feet per second.

This head of three feet can be given to the culvert in three ways:—

1st. The culvert having a level floor, the water will head up three feet on the upper side, the pipe being under pressure.

2d. A fall of three feet is given to the floor of the culvert in its length of 200 feet.

3d. A less fall than three feet is given to the floor in its length of 200 feet, and, in addition to this, sufficient heading takes place on the upper side to give the required velocity.

In so short a channel, an addition should be made to the head to generate such a high velocity as twelve feet per second, but as the flood water of the torrent arrives at the inlet of the syphon with a high velocity, a few inches additional to the head will suffice for this.

There is even a quicker method than that given above for finding the head approximately. For the same area of channel, a circle has the greatest hydraulic mean depth, and, therefore, requires the least head to give the same velocity. The culvert 6′x4′ has a cross-sectional area of twenty-four square feet, and a circular channel of the same area, will require less head to produce the same velocity.

We will use Table 56 for circular channels, with $n = .017$, to find the required head. In Table 56 the nearest area to twenty-four square feet is 23.758 square feet, having a diameter of five feet six inches. In the same line we find $c\sqrt{r} = 107.6$.

Let us now substitute the value of $c\sqrt{r}$ and v in formula (43).

$$\sqrt{s} = \frac{v}{c\sqrt{r}} = \frac{12}{107.6} = .111524$$

Now, n Table 33 the nearest value of $\sqrt{s}$ to this is .111803 opposite a slope of 1 in 80. As the length of the culvert is 200 feet, the head required for a circular channel is 2.5 feet, while that required for the rectangular channel 6′x4′ already found, is three feet.

EXAMPLE 23. *Given the diameter and grade of a Pipe to find the mean velocity and discharge by D'Arcy's formula (51), for clean cast-iron pipes.*

Humber, in his work on *Water Supply*, states:—

"With a 48-inch cast-iron pipe in the Lock Katrine Water Works, having an inclination of 1 in 1056, or five

feet per mile, the actual velocity was found to be 3.46 feet per second, and D'Arcy's formula gives practically the same results." Compute the velocity by the tables.

In Table 48, computed by D'Arcy's formula for clean pipes, we find opposite 4 feet diameter, that $a = 12.566$, $c\sqrt{r} = 112.6$, and $ac\sqrt{r} = 1414.7$.

We also find in Table 33 that opposite a grade of five feet per mile $\sqrt{s} = .030773$.

Let us now substitute value of $c\sqrt{r}$ and $\sqrt{s}$ in formula (41), and we have:—

$v = 112.6 \times .030773 = 3.46$ feet per second, being the same as the actual velocity, and also the same as the velocity obtained by computing by the longer method of D'Arcy's formula (51).

Now $Q = av = 12.566 \times 3.46 = 43.478$ cubic feet per second.

As a check on this, let us substitute the value of $ac\sqrt{r}$ and $\sqrt{s}$ in formula (45) and we have:—

$Q = 1414.7 \times .030773 = 43.535$ cubic feet per second, being practically the same as that found before.

EXAMPLE 24. *Given the grade, mean velocity and value of n, of a Circular Sewer to find its diameter.*

The grade of a circular sewer is to be 1 in 480, its mean velocity 4 feet per second, and its value of $n = .015$. What is the required diameter?

In Table 33 we find opposite a slope of 1 in 480 that $\sqrt{s} = .045644$—substitute this and the value of v, already given in formula (42).

$$c\sqrt{r} = \frac{v}{\sqrt{s}} \text{ and we have}$$

$$c\sqrt{r} = \frac{4}{.045644} = 87.63$$

Now look out in Table 55 for the nearest value of $c\sqrt{r}$ to this, which we find to be 87.15 opposite three feet four inches in diameter, which is the diameter required.

EXAMPLE 25. *Given the discharge, grade and value of n of a Circular Sewer to find its diameter.*

A circular brick sewer, with a value of $n = .015$, is to discharge 9 cubic feet per second and to have a grade of 1 in 200. What must its internal diameter be?

In Table 33, opposite a slope of 1 in 200, we find $\sqrt{s} = .07071$. Now substitute this value and also the value of Q already given in formula (47):—

$$ac\sqrt{r} = \frac{Q}{\sqrt{s}} \text{ and we have}$$

$$ac\sqrt{r} = \frac{9}{.07071} = 127.28$$

In Table 55, with a value of $n = .015$, the value of $ac\sqrt{r}$, nearest to this we find to be 130.58 opposite to which is the diameter of 1 foot and 9 inches which is the diameter required.

EXAMPLE 26. *Given the diameter, the value of n and the mean velocity in a Pipe, to find its inclination or grade.*

A sheet-iron, double riveted pipe, 18 inches diameter, with a very smooth interior, and laid in an almost straight line, is to have a velocity of 3 feet per second. Under the above favorable conditions its value of n is assumed equal to .011. What should its slope or grade be by Kutter's formula?

In Table 52, with a value of $n = .011$ the value of $c\sqrt{r}$ opposite a diameter of 1 foot 6 inches is 71.08. Substitute this value, and also the value of v already given, in formula (43):—

$$\sqrt{s} = \frac{v}{c\sqrt{r}} \text{ and we have}$$

$$\sqrt{s} = \frac{3}{71.08} = .042206$$

Look out the nearest value of $\sqrt{s}$ to this in Table 33, and we find it to be .042258 opposite a slope of 1 in 560. This is near enough for all practical purposes. If, however, a greater degree of accuracy is required, we have:—

$$\sqrt{s} = .042206 \text{ squaring each side}$$

$$s = .001781346436,$$

and $\frac{1}{s} = 561$. Therefore the slope is 1 in 561.

EXAMPLE 27. *Given the diameter, discharge and value of n of a Circular Conduit flowing full to find the slope or grade.*

A circular conduit flowing full is to have a diameter of 6 feet, and its value of n is assumed as equal to .017. What must be its slope or grade in order that its discharge may be 180 cubic feet per second?

In Table 56, with $n = .017$, we find opposite 6 feet in diameter that $ac\sqrt{r} = 3232.5$. Substitute this value and also the value of Q in formula (48), and we have:—

$$\sqrt{s} = \frac{Q}{ac\sqrt{r}} = \frac{180}{3232.5} = .055684$$

In Table 33 the nearest value of $\sqrt{s}$ to this is .055470 opposite a slope of 1 in 325. The required slope is, therefore, 1 in 325.

EXAMPLE 28. *To find the diameter in three sections of an Intercepting Sewer, with increasing discharge, the grade or inclination being the same throughout, and the value of n being given.*

A circular brick sewer has, for 500 feet of its length to

discharge, flowing full, 10 cubic feet per second, then for 600 feet more it has to discharge 12 cubic feet per second, and again, for 700 feet further, it has to discharge 15 cubic feet per second. The total fall available is 5 feet. Its value of $n = .015$. What is the required diameter and fall of each section?

In the total length of 1,800 feet there is a fall of 5 feet, that is at the rate of 1 in 360. In Table 33, opposite a fall of 1 in 360, we find $\sqrt{s} = .052705$.

By formula (47):— $ac\sqrt{r} = \dfrac{Q}{\sqrt{s}}$

In this equation substitute the values of Q and s for each section and compute the corresponding values of $ac\sqrt{r}$. Now, in the first column of Table 55, with $n = .015$, and opposite these values of $ac\sqrt{r}$ we shall find the diameters required. For example:—

$$\left.\begin{array}{l} ac\sqrt{r} = \dfrac{10}{.052705} = 189.7 \\[2ex] ac\sqrt{r} = \dfrac{12}{.052705} = 227.7 \\[2ex] ac\sqrt{r} = \dfrac{15}{.052705} = 284.6 \end{array}\right\} \begin{array}{c}\text{Opposite which in Table} \\ \text{55 is}\end{array} \left\{\begin{array}{l} \text{diam. } 2'\ 0'' \\[2ex] \text{diam. } 2'\ 2'' \\[2ex] \text{diam. } 2'\ 4'' \end{array}\right.$$

Now $s = \dfrac{h}{l}$ $\therefore$ $h = sl$, therefore, the

Fall of first section $= sl = .002777 \times 500 \ldots\ldots = 1.39$ ft.
Fall of second section $= sl = .002777 \times 600 \ldots = 1.67$ ft.
Fall of third section $= sl = .002777 \times 700 \ldots = 1.95$ ft.

Total fall .... 5.00 ft.

We have, therefore,

1st section, diameter 2′ 0″, fall 1.39 ft.
2d section, diameter 2′ 2″, fall 1.67 ft.
3d section, diameter 2′ 4″, fall 1.94 ft.

Example 29. *To find the value of c and n of a Pipe.*

A tuberculated pipe originally twenty-four inches in diameter, but reduced by tuberculation to a mean diameter in the clear of twenty-three inches, and having a slope of 1 in 1000, is found to discharge 4.5 cubic feet per second. What is its value of c and n?

$$v = \frac{Q}{a} = \frac{4.5}{2.885} = 1.56 \text{ feet per second.}$$

In Table 33 it will be found that a slope of 1 in 1000 has $\sqrt{s} = .031623$, and in Table 47 opposite, a diameter of twenty-three inches the value of $r = .479$, therefore $\sqrt{r} = .69$. Substitute values of v, $\sqrt{s}$ and $\sqrt{r}$ in formula (50).

$$c = \frac{v}{\sqrt{r} \times \sqrt{s}} \text{ and we have}$$

$$c = \frac{1.56}{.69 \times .031623} = 71.5$$

Now let us look in the tables of the values of c and $c\sqrt{r}$, and under a slope of 1 in 1000, and opposite $\sqrt{r} = .7$ (which is the nearest given to .69), until we find, in Table 21, under a value of $n = .017$ that $c = 72.6$, but by the column of difference it should be .51 less, therefore, the value of $c = 72.09$ and $n = .017$.

Now, as a check on this, let us find in Table 56 with $n = .017$, and opposite a diameter 1 foot 11 inches, that $ac\sqrt{r} = 144$. Substitute this value and also the value of $\sqrt{s}$ given above, in formula (45), and we have:—

$$Q = ac\sqrt{r} \times \sqrt{s}$$
$$= 144 \times .031623$$
$$= 4.55 \text{ cubic feet per second,}$$ being near enough for all practical purposes.

EXAMPLE 30. *Given the diameter of an old pipe to find the diameter of a new pipe to discharge double that of the old pipe.*

An old cast-iron pipe 3 feet 6 inches in diameter, whose natural co-efficient is assumed = .013, is to be replaced by a new sheet-iron pipe capable of discharging double that of the old pipe, the slope remaining unchanged. What is the diameter by Kutter's formula of the new pipe? It is to be dipped in hot asphalt, and its natural co-efficient is assumed = .011

Find by inspection in Table 54, with $n = .013$, the value of $ac\sqrt{r}$ opposite 3 feet 6 inches diameter, and it is found to be 1021.1. Then $1021.1 \times 2 = 2042.2$. As the value of n for the new pipe = 011, look out in Table 52 the value of $ac\sqrt{r}$ nearest to 2042.2 and it is found to be 2072.7 opposite a diameter of 4 feet 3 inches, which is the diameter required.

EXAMPLE 31. *Given the discharges and grades of a System of Pipes to find the diameters.*

A system of pipes consisting of one main and two branches, is required to discharge by one branch 15, and by another 24 cubic feet of water per minute, and, therefore, the main is to discharge 39 cubic feet of water per minute. The levels show the main pipe to have an inclination of 4 feet in 1000 feet, the first branch 3 feet in 600 feet, and the second branch 1 foot in 200 feet. What should be the diameters of the pipe?

The pipe being clean cast-iron pipe, Table 48, derived from D'Arcy's formula (51), will be used in the solution of the problem.

The main is to discharge 39 cubic feet per minute, equivalent to 0.65 cubic feet per second, with a grade of 1 in 250. One branch is to discharge 15 cubic feet per

minute, equivalent to 0.25 cubic feet per second, with a grade of 1 in 200, and the other branch 24 cubic feet per minute, equivalent to 0.4 cubic feet per second, with a grade of 1 in 200.

By inspection, we find in Table 33, that with a grade of 1 in 250 the $\sqrt{s} = .063246$ and a slope of 1 in 200 has $\sqrt{s} = .07071$.

Now, by formula (47):–

$$ac\sqrt{r} = \frac{Q}{\sqrt{s}} \therefore \text{ for main pipe}$$

$$ac\sqrt{r} = \frac{0.65}{.063246} = 10.277, \text{ and the}$$

nearest value of $ac\sqrt{r}$ to this, in Table 48, is 10.852, opposite to which is the diameter, 7 inches.

In the same way for the first branch

$$ac\sqrt{r} = \frac{0.25}{.07071} = 3.535, \text{ and the nearest value}$$

of $ac\sqrt{r}$ to this is 4.561, corresponding to diameter of 5 inches.

For the second branch:—

$$ac\sqrt{r} = \frac{0.4}{.07071} = 5.657, \text{ and the}$$

nearest value of $ac\sqrt{r}$ to this, in Table 48, is 7.3 opposite a diameter of 6 inches. The required diameters are, therefore, for the main pipe 7 inches, for the first branch 5 inches, and for the second branch 6 inches.

Although the explanation of this example in the use of the tables may appear somewhat long, still the actual work can be done very rapidly and with little trouble. If a comparison is made of the work required for the solution of this example, as given above, with the work required for its solution by the method of approximation as given in Weisbach's *Mechanics of Engineering*, from

which the example is extracted, it will be seen that there is a great saving of labor effected by the use of the tables.

EXAMPLE 32. *To find the dimensions of an Egg-shaped Sewer to replace a Circular Sewer.*

A circular sewer 5 feet in diameter and 4,800 feet in length has a fall of 16 feet. It is to be removed and replaced by an egg-shaped sewer with a fall of 8 feet, whose discharge flowing full shall equal that of the circular sewer flowing full, the value of n in each sewer being assumed $= .015$.

A grade of 16 in 4800 = 1 in 300, and in Table 33 the $\sqrt{s}$ corresponding to this is, .057735. In Table 55, opposite 5 feet diameter, the value of $ac\sqrt{r} = 2272.7$. Substitute this value and also the value of $\sqrt{s}$ in formula (45), and we have:—

$Q = 2272.7 \times .057735 = 131.21$ cubic feet per second, the discharge of the circular sewer. The egg-shaped sewer is to have a grade of 8 in 4800 = 1 in 600, and in Table 33 the $\sqrt{s}$ corresponding to this $= .040825$. Substitute this value and also the value of $\sqrt{s}$ just found in formula (47), and we have:—

$$ac\sqrt{r} = \frac{Q}{\sqrt{s}} = \frac{131.21}{.040825} = 3213.9$$

In Table 65 the nearest value of $ac\sqrt{r}$ to this is 3353, opposite an egg-shaped sewer having the dimensions of 4′ 10″ × 7′ 3″, therefore, with a value of $n = .015$ for both sewers.

A circular sewer of 5 feet in diameter, and having a grade of 1 in 300, has the same discharging capacity as an egg-shaped sewer 4′ 10″ × 7′ 3″, having a grade of 1 in 600.

EXAMPLE 33. *To find the diameter of a Circular Sewer whose discharge flowing full depth shall equal that of an Egg-shaped Sewer flowing one-third full depth.*

Find the diameter of a circular sewer, with $n = .013$, whose discharge flowing full shall equal that of the egg-shaped sewer in last example, flowing one-third full with $n = .015$, the slope being the same in each.

In Table 67 with $n = .015$ and $\frac{1}{3}$ full depth and opposite the size 4′ 10″ × 7′ 3″ we find $ac\sqrt{r} = 657.53$. Also in Table 54 circular $n = .013$, the nearest value of $ac\sqrt{r}$ to this is found to be 674.09 opposite a diameter of 3 feet, which is the diameter of the circular sewer required.

EXAMPLE 34. *In the same way as in Example 33, we can find the diameter of a Circular Sewer whose velocity flowing full shall equal the velocity of an Egg-shaped Sewer flowing one-third full depth.*

EXAMPLE 35. *To find the dimensions and grade of an Egg-shaped Sewer flowing full, the mean velocity and discharge being given.*

An egg-shaped sewer flowing full is to have a mean velocity not greater than five feet per second, and is to discharge 108 cubic feet per second. Its value of n is .015. What are its dimensions and grade?

By formula (46).

$$a = \frac{Q}{v} = \frac{108}{5} = 21.6 \text{ square feet.}$$

In column two of Table 65, the nearest area to this is 21.566 square feet opposite the dimensions 4′ 4″ × 6′ 6″. In the same line we find the value of $c\sqrt{r} = 116.0$, and $ac\sqrt{r} = 2501.4$. Substitute this latter value and the value of Q in formula 48, and we have:—

$$\sqrt{s} = \frac{Q}{ac\sqrt{r}} = \frac{108}{2501.4} = .043176, \text{ and in table 33.}$$

the nearest to this, is .043234 opposite a slope of 1 in 535. The sewer required is therefore 4′ 4″ × 6′ 6″, and has a slope of 1 in 535.

As a *check* on this work by formula 45, and by substituting the values of a, $c\sqrt{r}$ and $\sqrt{s}$ already found, we have:—

$$Q = a \times c\sqrt{r} \times \sqrt{s}$$
$$= 21.6 \times 116 \times .043234$$
$= 108.3$ cubic feet per second, being near enough for all practical purposes.

EXAMPLE 36. *The diameter and grade of a Circular Sewer being given, to find the dimensions and grade of an Egg-shaped Sewer, whose discharge flowing two-thirds full depth shall equal that of the Circular Sewer flowing full depth, and whose mean velocity at the same depth shall not exceed a certain rate.*

A circular sewer 6 feet in diameter and with a slope of 1 in 600 is to be removed and to be replaced by an egg-shaped sewer whose discharge flowing at two-thirds of its full depth, shall be equal to that of the circular sewer flowing full depth, and whose mean velocity at the same two-thirds depth shall not exceed five feet per second, the value n in each being $= .015$. Give the dimensions and slope of the egg-shaped sewer.

In Table 55 for circular channels with $n = .015$ and 6 feet in diameter, the value of $ac\sqrt{r} = 3702.3$, and in Table 33 opposite 1 in 600, the value of $\sqrt{s} = .040825$. Substitute these values in formula (45).

$Q = ac\sqrt{r} \times \sqrt{s}$ and we get

$Q = 3702.3 \times .040825 = 151.15$ cubic feet per second as the discharge of the circular sewer. Now

substitute this discharge and the velocity given, five feet per second, in formula (46).

$$a = \frac{Q}{v} \text{ and we get}$$

$$a = \frac{151.15}{5} = 30.23 \text{ square feet, the}$$

area at two-thirds full depth of the egg-shaped sewer.

In Table 66, of egg-shaped sewers flowing two-thirds full depth with $n = .015$, we find the nearest value of a to this is 30.317 square feet opposite a sewer having the dimensions of 6 feet 4 inches by 9 feet 6 inches. At the same time take out the value of $ac\sqrt{r}$ in the same line and we find it equal to 4811.9. Substitute this value of $ac\sqrt{r}$, and also the value of Q, already found in formula (48),

$$\sqrt{s} = \frac{Q}{ac\sqrt{r}} \text{ and we get}$$

$$\sqrt{s} = \frac{151.15}{4811.9} = .031412.$$

Look out in Table 33, and by interpolation we find the nearest slope to this is 1 in 1015. The egg-shaped sewer required is, therefore, 6′ 4″ × 9′ 6″ and the grade 1 in 1015.

EXAMPLE 37. *To find the dimensions and grade of an Egg-shaped Sewer to have a certain discharge when flowing full, and whose mean velocity shall not exceed a certain rate when flowing two-thirds full depth.*

An egg-shaped sewer is to discharge 110 cubic feet per second flowing full, and its mean velocity flowing two-thirds full depth is not to exceed 5 feet per second. Find its dimensions and slope, the value of n being taken $= .015$.

In an egg-shaped sewer the velocity flowing full is always less than the velocity flowing two-thirds full, therefore, as a first approximation let us assume the velocity flowing full at 5 feet per second.

$$a = \frac{Q}{v} = \frac{110}{5} = 22 \text{ square feet, the area of the}$$

assumed egg-shaped sewer flowing full, and in Table 65 the nearest size sewer to this is 4′ 4″ × 6′ 6″. Now with these dimensions the value of $c\sqrt{r}$ full depth = 116.0 and Table 66 the value of $c\sqrt{r}$ two-thirds full depth = 123.1; therefore, we may assume that the velocity of the sewer of the given dimensions flowing full is about six per cent. less than when flowing two-thirds full depth, that is, assuming the velocity at two-thirds full depth = 5 feet per second the velocity at full depth will be about 4.7 feet per second. Substituting this velocity and also the given discharge in formula (46),

$$a = \frac{Q}{v} = \frac{110}{4.7} = 23.4 \text{ square feet, the area of}$$

egg-shaped sewer flowing full. In Table 65, the nearest size opposite to this area is 4′ 6″ × 6′ 9″ which is the diameter required for the egg-shaped sewer. At the same time that this size of sewer is found, its value of $ac\sqrt{r}$ will be found on the same line = 2770. Substitute this value and also the value of Q in formula (48), and we have:—

$$\sqrt{s} = \frac{Q}{ac\sqrt{r}} = \frac{110}{2770} = .039711.$$

Look out in Table 33 and the nearest $\sqrt{s}$ to this is .039684 opposite a slope of 1 in 635. Therefore, the size of the sewer is 4′ 6″ × 6′ 9″, and its grade 1 in 635. As a check on the above work by substituting the factors already found, we can find the discharge of the sewer

flowing full depth, and also find the mean velocity of the same sewer flowing two-thirds full depth.

$Q = ac\sqrt{r} \times \sqrt{s} = 2770 \times .039684 = 109.9$ cubic feet per second, that is, practically, 110 cubic feet per second which was required, and

$v = c\sqrt{r} \times \sqrt{s} = 126.3 \times .039684 = 5.01$ feet per second, that is, practically 5 feet per second, which was required.

TABLE 47.

Giving the value of the hydraulic mean depth r, for Circular Pipes, Conduits and Sewers. The hydraulic mean depth is equal to one-fourth the diameter of a circular channel.

Diameter, ft.	in.	r in feet.	Diameter, ft.	in.	r in feet.	Diameter, ft.	in.	r in feet.	Diameter, ft.	in.	r in feet.
	⅜	.0078	2	1	.521	4	7	1.146	9	3	2.312
	½	.0104	2	2	.542	4	8	1.167	9	6	2.375
	¾	.0156	2	3	.562	4	9	1.187	9	9	2.437
	1	.0208	2	4	.583	4	10	1.208	10		2.5
	1¼	.0260	2	5	.604	4	11	1.229	10	3	2.562
	1½	.0312	2	6	.625	5		1.25	10	6	2.625
	1¾	.0364	2	7	.646	5	1	1.271	10	9	2.687
	2	.0417	2	8	.667	5	2	1.292	11		2.750
	2½	.052	2	9	.687	5	3	1.312	11	3	2.812
	3	.063	2	10	.708	5	4	1.333	11	6	2.875
	4	.084	2	11	.729	5	5	1.354	11	9	2.937
	5	.104	3		.75	5	6	1.375	12		3.
	6	.125	3	1	.771	5	7	1.396	12	3	3.062
	7	.146	3	2	.792	5	8	1.417	12	6	3.125
	8	.167	3	3	.812	5	9	1.437	12	9	3.187
	9	.187	3	4	.833	5	10	1.558	13		3.25
	10	.208	3	5	.854	5	11	1.479	13	3	3.312
	11	.229	3	6	.875	6		1.5	13	6	3.375
1		.250	3	7	.896	6	3	1.562	13	9	3.437
1	1	.271	3	8	.917	6	6	1.625	14		3.5
1	2	.292	3	9	.937	6	9	1.687	14	6	3.625
1	3	.313	3	10	.958	7		1.75	15		3.75
1	4	.333	3	11	.979	7	3	1.812	15	6	3.875
1	5	.354	4		1.	7	6	1.879	16		4.
1	6	.375	4	1	1.021	7	9	1.937	16	6	4.125
1	7	.396	4	2	1.042	8		2.	17		4.250
1	8	.417	4	3	1.062	8	3	2.062	17	6	4.375
1	9	.437	4	4	1.083	8	6	2.125	18		4.5
1	10	.458	4	5	1.104	8	9	2.187	19		4.75
1	11	.479	4	6	1.125	9		2.25	20		5.
2		.5									

TABLE 48.

Circular Pipes, Conduits, etc., flowing under pressure. Based on D'Arcy's formula for the flow of water through clean cast-iron pipes.

Table giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only for clean cast-iron pipes, flowing under pressure, and also for other pipes or conduits having surfaces of other material equally rough.

d = diameter in ft.	d = diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	d = diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	3/8	.00077	5.251	.00403	1	10	2.640	75.32	198.83
	1/2	.00136	6.702	.00914	1	11	2.885	77.05	222.30
	3/4	.00307	9.309	.02855	2		3.142	78.80	247.57
	1	.00545	11.61	.06334	2	1	3.409	80.53	274.53
	1¼	.00852	13.68	.11659	2	2	3.687	82.15	302.90
	1½	.01227	15.58	.19115	2	3	3.976	83.77	333.07
	1¾	.01670	17.32	.28936	2	4	4.276	85.39	365.14
	2	.02182	18.96	.41357	2	5	4.587	86.89	398.57
	2½	.0341	21.94	.74786	2	6	4.909	88.39	433.92
	3	.0491	24.63	1.2089	2	7	5.241	90.01	471.73
	4	.0873	29.37	2.5630	2	8	5.585	91.51	511.10
	5	.136	33.54	4.5610	2	9	5.939	92.90	551.72
	6	.196	37.28	7.3068	2	10	6.305	94.40	595.17
	7	.267	40.65	10.852	2	11	6.681	95.78	639.88
	8	.349	43.75	15.270	3		7.068	97.17	686.76
	9	.442	46.73	20.652	3	1	7.466	98.55	735.75
	10	.545	49.45	26.952	3	2	7.875	99.93	786.94
	11	.660	52.16	34.428	3	3	8.295	101.2	839.38
1		.785	54.65	42.918	3	4	8.726	102.6	895.07
1	1	.922	57.	52.551	3	5	9.169	103.8	952.10
1	2	1.069	59.34	63.435	3	6	9.621	105.1	1011.2
1	3	1.227	61.56	75.537	3	7	10.084	106.4	1072.6
1	4	1.396	63.67	88.886	3	8	10.559	107.6	1136.5
1	5	1.576	65.77	103.66	3	9	11.044	108.9	1202.7
1	6	1.767	67.75	119.72	3	10	11.541	110.2	1271.4
1	7	1.969	69.74	137.31	3	11	12.048	111.4	1342.4
1	8	2.182	71.71	156.46	4		12.566	112.6	1414.7
1	9	2.405	73.46	176.66	4	1	13.096	113.7	1489.4

TABLE 48.

Circular Pipes, Conduits, etc., flowing under pressure. Based on D'Arcy's formula for the flow of water through clean cast-iron pipes, for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
4	2	13.635	115.	1567.8	8	6	56.745	165.	9364.7
4	3	14.186	116.1	1647.6	8	9	60.132	167.4	10068.
4	4	14.748	117.3	1729.8	9		63.617	169.8	10804.
4	5	15.321	118.4	1814.6	9	3	67.201	172.2	11575.
4	6	15.904	119.6	1901.9	9	6	70.882	174.5	12370.
4	7	16.499	120.6	1990.1	9	9	74.662	176.8	13200.
4	8	17.104	121.8	2082.6	10		78.540	179.1	14066.
4	9	17.721	122.8	2176.1	10	3	82.516	181.4	14967.
4	10	18.348	124.	2274.1	10	6	86.590	183.6	15893.
4	11	18.986	125.1	2374.8	10	9	90.763	185.7	16856.
5		19.635	126.1	2476.4	11		95.033	187.9	17855.
5	1	20.295	127.2	2580.5	11	3	99.402	190.1	18892.
5	2	20.966	128.3	2689.9	11	6	103.869	192.2	19966.
5	3	21.648	129.3	2799.7	11	9	108.434	194.3	21065.
5	4	22.340	130.4	2912.4	12		113.098	196.3	22204.
5	5	23.044	131.4	3027.8	12	3	117.859	198.4	23379.
5	6	23.758	132.4	3146.3	12	6	122.719	200.4	24598.
5	7	24.484	133.4	3264.9	12	9	127.677	202.4	25840.
5	8	25.220	134.4	3388.9	13		132.733	204.4	27134.
5	9	25.967	135.4	3516.	13	3	137.887	206.4	28456.
5	10	26.725	136.4	3646.1	13	6	143.139	208.3	29818.
5	11	27.494	137.4	3776.2	13	9	148.490	210.2	31219.
6		28.274	138.4	3912.8	14		153.938	212.2	32664.
6	3	30.680	141.3	4333.6	14	6	165.130	216.	35660.
6	6	33.183	144.1	4782.1	15		176.715	219.6	38807.
6	9	35.785	146.9	5255.1	15	6	188.692	223.3	42125.
7		38.485	149.6	5757.5	16		201.062	226.9	45621.
7	3	41.283	152.2	6284.6	16	6	213.825	230.4	49273.
7	6	44.179	154.9	6841.6	17		226.981	233.9	53082.
7	9	47.173	157.5	7429.3	17	6	240.529	237.3	57074.
8		50.266	160.	8043.	18		254.470	240.7	61249.
8	3	53.456	162.5	8688.	19		283.529	247.4	70154.
					20		314.159	253.8	79736.

TABLE 49.

Circular Pipes, Conduits, etc., flowing under pressure. Based on D'Arcy's formula for the flow of water through old cast-iron pipes lined with deposit.

Table giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only for old cast-iron pipes flowing under pressure, and also for other pipes or conduits having surfaces of other material equally rough.

Diameter in ft. in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft. in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
3/8	.00077	3.532	.00272	1 9	2.405	49.410	118.83
1/2	.00136	4.507	.00613	1 10	2.640	50.658	133.74
3/4	.00307	6.261	.01922	1 11	2.885	51.829	149.53
1	.00545	7.811	.04257	2	3.142	52.961	166.41
1 1/4	.00852	9.255	.07885	2 1	3.409	54.166	184.65
1 1/2	.01227	10.48	.12855	2 2	3.687	55.258	203.74
1 3/4	.01670	11.65	.19462	2 3	3.976	56.348	224.04
2	.02182	12.75	.27824	2 4	4.276	57.436	245.60
2 1/2	.0341	14.76	.50321	2 5	4.587	58.448	268.10
3	.0491	16.56	.81333	2 6	4.909	59.455	291.87
4	.0873	19.75	1.7246	2 7	5.241	60.544	317.31
5	.136	22.56	3.0681	2 8	5.585	61.55	343.8
6	.196	25.07	4.9147	2 9	5.939	62.49	371.1
7	.267	27.34	7.2995	2 10	6.305	63.49	400.3
8	.349	29.43	10.271	2 11	6.681	64.42	430.4
9	.442	31.42	13.891	3	7.068	65.35	461.9
10	.545	33.26	18.129	3 1	7.466	66.29	494.9
11	.660	35.09	23.158	3 2	7.875	67.21	529.3
1	.785	36.75	28.867	3 3	8.295	68.09	564.6
1 1	.922	38.33	35.345	3 4	8.726	69.	602.
1 2	1.069	39.91	42.668	3 5	9.169	69.85	640.4
1 3	1.227	41.41	50.811	3 6	9.621	70.70	680.2
1 4	1.396	42.83	59.788	3 7	10.084	71.55	721.5
1 5	1.576	44.24	69.723	3 8	10.559	72.40	764.5
1 6	1.767	45.57	80.531	3 9	11.044	73.25	809.
1 7	1.969	46.90	93.357	3 10	11.541	74.10	855.2
1 8	2.182	48.34	105.25	3 11	12.048	74.95	903.

TABLE 49.

Circular Pipes, Conduits, Sewers, etc., flowing under pressure. Based on D'Arcy's formula for the flow of water through old cast-iron pipes lined with deposit, for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

Diameter in ft.	Diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft.	Diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
4		12.566	75.73	951.6	8	6	56.745	111.	6299.1
4	1	13.096	76.50	1000.8	8	9	60.132	112.6	6772.2
4	2	13.635	77.35	1054.6	9		63.617	114.2	7267.3
4	3	14.186	78.12	1108.2	9	3	67.201	115.8	7785.2
4	4	14.748	78.89	1163.5	9	6	70.882	117.4	8320.6
4	5	15.321	79.66	1220.5	9	9	74.662	118.9	8879.
4	6	15.904	80.43	1279.2	10		78.540	120.4	9460.9
4	7	16.499	81.13	1338.6	10	3	82.516	122.	10067.
4	8	17.104	81.90	1400.8	10	6	86.590	123.4	10690.
4	9	17.721	82.20	1456.8	10	9	90.763	124.9	11338.
4	10	18.348	83.37	1529.6	11		95.033	126.3	12010.
4	11	18.986	84.14	1597.5	11	3	99.402	127.8	12707.
5		19.635	84.83	1665.7	11	6	103.869	129.3	13429.
5	1	20.295	85.54	1735.8	11	9	108.434	130.6	14160.
5	2	20.966	86.30	1809.3	12		113.098	132.	14035.
5	3	21.648	86.99	1883.2	12	3	117.859	133.4	15727.
5	4	22.340	87.69	1958.9	12	6	122.719	134.8	16545.
5	5	23.044	88.38	2036.6	12	9	127.677	136.1	17380.
5	6	23.758	89.07	2116.2	13		132.733	137.5	18252.
5	7	24.484	89.69	2191.5	13	3	137.887	138.8	19140.
5	8	25.220	90.38	2279.5	13	6	143.139	140.1	20056.
5	9	25 967	91.08	2365.	13	9	148.490	141.4	20999.
5	10	26.725	91.77	2452.9	14		153.938	142.7	21971.
5	11	27.494	92.39	2540.1	14	6	165.130	145.2	23986.
6		28.274	93.08	2631.7	15		176.715	147.7	26103.
6	3	30.680	95.	2914.8	15	6	188.692	150.1	28335.
6	6	33.183	96.93	3216.4	16		201.062	152.6	30686.
6	9	35.785	98.78	3534.7	16	6	213.825	155.	33144.
7		38.485	100.61	3872.5	17		226.981	157.3	35704.
7	3	41.283	102.41	4227.1	17	6	240.529	159.6	38389.
7	6	44.179	104.11	4601.9	18		254.470	161.9	41199.
7	9	47.173	105.91	4997.2	19		283.529	166.4	47186.
8		50.266	107.61	5409.9	20		314.159	170.7	53633.
8	3	53.456	109.31	5843.6					

TABLE 50.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula, with $n = .009$.

Table giving the values of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only when the value of n, that is the coefficient of roughness of lining of channel = .09, as for well-planed timber in perfect order and alignment; otherwise, perhaps .01 would be suitable. It is also suitable for other channels having surfaces equally rough.

Diameter in ft.	Diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft.	Diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	35.31	4.803	2	6	4.909	128.8	622.3
	6	.196	40.62	7.962	2	7	5.241	131.9	691.3
	7	.267	45.70	12.20	2	8	5.585	134.7	752.2
	8	.349	50.55	17.64	2	9	5.939	137.3	815.3
	9	.442	55.13	24.37	2	10	6.305	140.1	883.4
	10	.545	59.49	32.42	2	11	6.681	142.7	953.7
	11	.660	64.	42.24	3		7.068	145.4	1027.6
1		.785	68.25	53.60	3	1	7.466	148.1	1105.5
1	1	.922	72.11	66.49	3	2	7.875	150.7	1187.1
1	2	1.069	76.06	81.31	3	3	8.295	153.2	1270.9
1	3	1.227	79.90	98.03	3	4	8.726	155.8	1359.9
1	4	1.396	83.60	116.7	3	5	9.169	158.3	1451.3
1	5	1.576	87.38	137.7	3	6	9.621	160.7	1546.3
1	6	1.767	90.86	160.5	3	7	10.084	163.2	1645.4
1	7	1.969	94.34	185.7	3	8	10.559	165.6	1749.
1	8	2.182	97.86	213.5	3	9	11.044	168.1	1856.6
1	9	2.405	101.	242.9	3	10	11.541	170.6	1969.
1	10	2.640	104.4	275.7	3	11	12.048	173.1	2085.6
1	11	2.885	107.7	310.6	4		12.566	175.4	2204.1
2		3.142	110.9	348.4	4	1	13.096	177.6	2326.2
2	1	3.409	114.	388.7	4	2	13.635	180.1	2455.6
2	2	3.687	117.	431.5	4	3	14.186	182.3	2586.7
2	3	3.976	120.	477.3	4	4	14.748	184.6	2722.5
2	4	4.276	123.1	526.3	4	5	15.321	186.9	2863.
2	5	4.587	125.9	577.7	4	6	15.904	189.1	3008.2

TABLE 50.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula, with $n = .009$, for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
4	7	16.499	191.2	3154.6	9	3	67.201	295.7	19875
4	8	17.104	193.5	3309.5	9	6	70.882	300.4	21296
4	9	17.721	195.5	3465.6	9	9	74.562	305.1	22784
4	10	18.348	197.9	3630.6	10		78.540	309.9	24339
4	11	18.986	200.1	3799.9	10	3	82.516	314.6	25962
5		19.635	202.2	3969.8	10	6	86.590	319.1	27630
5	1	20.295	204.2	4144.7	10	9	90.763	323.5	29365
5	2	20.966	206.5	4329.5	11		95.033	328.	31171
5	3	21.648	208.5	4514.9	11	3	99.402	332.5	33051
5	4	22.340	210.6	4705.4	11	6	103.869	337.	35005
5	5	23.044	212.7	4901.1	11	9	108.434	341.3	37006
5	6	23.758	214.7	5102.4	12		113.098	345.5	39079
5	7	24.484	216.6	5303.7	12	3	117.859	349.8	41230
5	8	25.220	218.7	5515.9	12	6	122.719	354.1	43459
5	9	25.967	220.8	5733.7	12	9	127.677	358.2	45733
5	10	26.725	222.8	5956.	13		132.733	362.5	48117
5	11	27.494	224.7	6177.7	13	3	137.887	366.5	50537
6		28.274	226.7	6411.1	13	6	143.139	370.5	53036
6	3	30.680	232.5	7133.1	13	9	148.490	374.5	55619
6	6	33.183	238.3	790.7	14		153.938	378.6	58280
6	9	35.785	243.9	8728.	14	6	165.130	386.4	63805
7		38.485	249.4	9599.6	15		176.715	394.1	69639
7	3	41.283	254.7	10517.	15	6	188.692	401.7	75799
7	6	44.179	260.1	11492.	16		201.062	409.4	82315
7	9	47.173	265.5	12525.	16	6	213.825	416.7	89114
8		50.266	270.6	13605.	17		226.981	423.9	96219
8	3	53.456	275.8	14741.	17	6	240.529	431.1	103687
8	6	56.745	280.9	15941.	18		254.470	438.2	111519
8	9	60.132	285.9	17190.	19		283.529	452.3	128254
9		63.617	290.8	18503.	20		314.159	465.7	146322

TABLE 51.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula, with $n = .010$.

Table giving the values of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .010$, as for plaster in pure cement; planed timber; glazed, coated or enamelled stoneware and iron pipes; glazed surfaces of every sort in perfect order, and also surfaces of other material equally rough.

Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	30.54	4.154	2	6	4.909	114.	559.6
	6	.196	35.23	6.906	2	7	5.241	116.8	612.
	7	.267	39.73	10.61	2	8	5.585	119.3	668.3
	8	.349	44.02	15.36	2	9	5.939	121.6	722.4
	9	.442	48.09	21.25	2	10	6.305	124.2	783.1
	10	.545	51.96	28.32	2	11	6.681	126.6	845.8
	11	.660	55.97	36.94	3		7.068	129.	911.8
1		.785	59.75	46.93	3	1	7.466	131.4	981.2
1	1	.922	63.19	58.26	3	2	7.875	133.8	1054.1
1	2	1.069	66.71	71.31	3	3	8.295	136.1	1128.9
1	3	1.227	70.13	86.05	3	4	8.726	138.5	1208.4
1	4	1.396	73.44	102.5	3	5	9.169	140.7	1289.9
1	5	1.576	76.81	121.	3	6	9.621	142.9	1374.7
1	6	1.767	79.93	141.2	3	7	10.084	145.1	1463.3
1	7	1.969	83.05	163.5	3	8	10.559	147.3	1555.8
1	8	2.182	86.21	188.1	3	9	11.044	149.6	1652.1
1	9	2.405	89.05	214.1	3	10	11.541	151.8	1752.5
1	10	2.640	92.19	243.3	3	11	12.048	154.1	1856.9
1	11	2.885	95.03	274.2	4		12.566	156.2	1962.8
2		3.142	97.91	307.6	4	1	13.096	158.2	2072.
2	1	3.409	100.7	343.4	4	2	13.635	160.4	2187.7
2	2	3.687	103.4	381.3	4	3	14.186	162.5	2305.
2	3	3.976	106.1	421.9	4	4	14.748	164.5	2426.5
2	4	4.276	108.8	465.4	4	5	15.321	166.6	2552.2
2	5	4.587	111.41	511.	4	6	15.904	168.6	2682.1

TABLE 51.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula, with $n = .010$, for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
4	7	16.499	170.5	2813.2	9	3	67.201	265.4	1783.9
4	8	17.104	172.6	2951.9	9	6	70.882	269.7	19118.
4	9	17.721	174.5	3091.8	9	9	74.662	274.	20157.
4	10	18.348	176.6	3238.7	10		78.540	278.3	21858.
4	11	18.986	178.6	3391.	10	3	82.516	282.6	23320.
5		19.635	180.4	3543.	10	6	86.590	286.7	24823.
5	1	20.295	182.3	3699.6	10	9	90.763	290.7	26390.
5	2	20.966	184.3	3865.1	11		95.033	294.8	28020.
5	3	21.648	186.2	4031.1	11	3	99.402	298.9	29717.
5	4	22.340	188.1	4202.	11	6	103.87	303.1	31482.
5	5	23.044	189.9	4377.5	11	9	108.43	306.9	33285.
5	6	23.758	191.8	4557.8	12		113.10	310.8	35156.
5	7	24.484	193.5	4738.1	12	3	117.86	314.7	37095.
5	8	25.220	195.4	4928.2	12	6	122.72	318.6	39104.
5	9	25.967	197.3	5123.5	12	9	127.68	322.3	41157.
5	10	26.725	199.2	5323.	13		132.73	326.3	43307.
5	11	27.494	200.8	5521.7	13	3	137.88	329.9	45493.
6		28.274	202.7	5731.5	13	6	143.14	333.6	47751.
6	3	30.680	207.9	6379.5	13	9	148.49	337.3	50085.
6	6	33.183	213.2	7075.2	14		153.94	341.	52491.
6	9	35.785	218.3	7812.7	14	6	165.13	348.2	57496.
7		38.485	223.3	8595.1	15		176.72	355.1	62748.
7	3	41.283	228.2	9420.3	15	6	188.69	362.	68313.
7	6	44.179	233.	10296.	16		201.06	369.	74191.
7	9	47.173	237.9	11225.	16	6	213.83	375.7	80342.
8		50.266	242.6	12196.	17		226.98	382.3	86769.
8	3	53.456	247.3	13219.	17	6	240.53	388.8	93528.
8	6	56.745	252.	14298.	18		254.47	395.4	100617.
8	9	60.132	256.5	15422.	19		283.53	408.3	115769.
9		63.617	261.	16604.	20		314.16	420.6	132133.

TABLE 52.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula, with $n = .011$.

Table giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .011$, as for surfaces carefully plastered with cement with one-third sand, in good condition; also for iron, cement and terra-cotta pipes, well jointed and in best order, and also surfaces of other material equally rough.

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	26.76	3.6398	2	11	6.681	113.5	758.16
	6	.196	30.93	6.0627	3		7.068	115.7	817.50
	7	.267	34.94	9.3294	3	1	7.466	117.9	880.03
	8	.349	38.77	13.531	3	2	7.875	120.1	945.69
	9	.442	42.40	18.742	3	3	8.295	122.1	1013.1
	10	.545	45.83	24.976	3	4	8.726	124.3	1084.6
	11	.660	49.46	32.644	3	5	9.169	126.3	1158.
1		.785	52.85	41.487	3	6	9.621	128.3	1234.4
1	1	.922	55.95	51.588	3	7	10.084	130.3	1314.1
1	2	1.069	59.13	63.210	3	8	10.559	132.3	1397.4
1	3	1.227	62.22	76.347	3	9	11.044	134.4	1484.2
1	4	1.396	65.21	91.037	3	10	11.541	136.4	1574.7
1	5	1.576	68.26	107.58	3	11	12.048	138.3	1666.5
1	6	1.767	71.08	125.60	4		12.566	140.4	1764.3
1	7	1.969	73.90	145.51	4	1	13.096	142.2	1862.7
1	8	2.182	76.76	167.50	4	2	13.635	144.3	1967.1
1	9	2.405	79.33	190.79	4	3	14.186	146.1	2072.7
1	10	2.640	82.11	216.76	4	4	14.748	148.	2182.5
1	11	2.885	84.75	244.50	4	5	15.321	149.9	2296.
2		3.142	87.36	274.50	4	6	15.904	151.7	2413.3
2	1	3.409	89.94	306.60	4	7	16.499	153.4	2531.7
2	2	3.687	92.38	340.59	4	8	17.104	155.3	2657.1
2	3	3.976	94.84	377.07	4	9	17.721	157.1	2783.4
2	4	4.276	97.33	416.16	4	10	18.348	159.	2917
2	5	4.587	99.66	457.13	4	11	18.986	160.9	3054.1
2	6	4.909	102.	500.78	5		19.635	162.6	3191.8
2	7	5.241	104.5	547.92	5	1	20.295	164.5	3337.5
2	8	5.585	106.8	596.70	5	2	20.966	166.	3480.8
2	9	5.939	109.	647.18	5	3	21.648	167.9	3634.2
2	10	6.305	111.3	701.77	5	4	22.340	169.6	3789.

TABLE 52.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .011$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
5	5	23.044	171.3	3944.4	10	9	90.763	264.	23951
5	6	23.758	173.1	4111.9	11		95.033	567.7	25444
5	7	24.484	174.6	4275.4	11	3	99.402	271.5	26987
5	8	25.220	176.4	4448.	11	6	103.869	275.3	28593
5	9	25.967	178.1	4625.2	11	9	108.434	278.8	30235
5	10	26.725	179.8	4806.1	12		113.098	282.4	31937
5	11	27.494	181.4	4986.1	12	3	117.359	286.	33702
6		28.274	183.1	5176.3	12	6	122.719	289.5	35529
6	3	30.680	187.9	5764.	12	9	127.677	292.9	37399
6	6	33.183	192.7	6394.9	13		132.733	296.5	39358
6	9	35.785	197.2	7057.1	13	3	137.887	299.9	41352
7		38.485	202.	7774.3	13	6	143 139	303.3	43412
7	3	41.283	206.5	8522.9	13	9	148.490	306.7	45543
7	6	44.179	210.9	9318.3	14		153.938	310.1	47739
7	9	47.173	215.4	10162.	14	6	165.130	316.8	52308
8		50.266	219.7	11044.	15		176.715	323.1	57103
8	3	53.456	224.	11978.	15	6	188.692	329.6	62186
8	6	56.745	228.3	12954.	16		201.062	336.	67557
8	9	60.132	232.4	13974.	16	6	213.825	342.2	73176
9		63.617	236.6	15049.	17		226.981	348.3	79050
9	3	67.201	240.7	16173.	17	6	240.529	354.3	85229
9	6	70.882	244.6	17338	18		254.470	360 4	91711
9	9	74.662	248.6	18558.	19		283.529	372.3	105570
10		78 540	252.5	19834	20		314.159	383.8	120570
10	3	82.516	256.5	21166.					
10	6	86.590	260.2	22534.					

TABLE 53.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .012$.

Table giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .012$ as for unplaned timber when perfectly continuous on the inside and also flumes, and the surfaces of other material equally rough.

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	23.70	3.2234	2	4	4.276	87.81	375.46
	6	.196	27.45	5.3800	2	5	4.587	89.94	412.54
	7	.267	31.05	8.2911	2	6	4.909	92.09	452.07
	8	.349	34.51	12.042	2	7	5.241	94.41	494.78
	9	.442	37.80	16.708	2	8	5.585	96.52	539.07
	10	.545	40.95	22.317	2	9	5.939	98.49	584.90
	11	.666	44.22	29.183	2	10	6.305	100.6	634.46
1		.785	47.30	37.149	2	11	6.681	102.6	685.64
1	1	.922	50.11	46.19	3		7.068	104.6	739.59
1	2	1.069	52.99	56.64	3	1	7.466	106.7	796.38
1	3	1.227	55.78	68.44	3	2	7.875	108.7	856.12
1	4	1.396	58.50	81.66	3	3	8.295	110.6	917.41
1	5	1.576	61.26	96.54	3	4	8.726	112.6	982.39
1	6	1.767	63.83	112.79	3	5	9.169	114.4	1049.1
1	7	1.969	66.41	130.76	3	6	9.621	116.3	1118.6
1	8	2.182	69.03	150.61	3	7	10.084	118.1	1191.1
1	9	2.405	71.38	171.66	3	8	10.559	120.	1267.
1	10	2.640	73.92	195.14	3	9	11.044	121.9	1345.9
1	11	2.885	76.33	220.21	3	10	11.541	123.8	1428.3
2		3.142	78.72	247.33	3	11	12.048	125.7	1514.
2	1	3.409	81.07	276.38	4		12.566	127.4	1600.9
2	2	3.687	83.29	307.10	4	1	13.096	129.1	1690.7
2	3	3.976	85.54	340.10	4	2	13.635	131.	1785.8

TABLE 53.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .012$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
4	3	14.186	132.7	1882.3	8	9	60.132	212.3	12766.
4	4	14.748	134.4	1982.3	9		63.617	216.2	13751.
4	5	15.321	136.2	2085.9	9	3	67.201	219.9	14780.
4	6	15.904	137.9	2193.	9	6	70.882	223.6	15847.
4	7	16.499	139.5	2301.	9	9	74.662	227.2	16965.
4	8	17.104	141.2	2415.4	10		78.540	230.9	18134.
4	9	17.721	142.8	2530.8	10	3	82.516	234.6	19356.
4	10	18.348	144.6	2652.8	10	6	86.590	238.	20612.
4	11	18.986	146.3	2777.8	10	9	90.763	241.5	21921.
5		19.635	147.9	2903.6	11		95.033	245.	23285.
5	1	20.295	149.4	3032.9	11	3	99.402	248.5	24703.
5	2	20.966	151.2	3169.8	11	6	103.869	252.	26179.
5	3	21.648	152.8	3307.	11	9	108.434	255.4	27689.
5	4	22.340	154.4	3448.3	12		113.098	258.7	29254.
5	5	23.044	155.9	3593.5	12	3	117.859	262.	30876.
5	6	23.758	157.5	3742.7	12	6	122.719	265.3	32558.
5	7	24.484	159.	3892.	12	9	127.677	268.5	34277.
5	8	25.220	160.6	4049.5	13		132.733	271.8	36077.
5	9	25.967	162.2	4211.2	13	3	137.887	274.9	37909.
5	10	26.729	163.8	4376.4	13	6	143.139	278.1	39802.
5	11	27.494	165.1	4540.5	13	9	148.490	281.2	41755.
6		28.274	166.7	4713.9	14		153.938	284.4	43773.
6	3	30.680	171.1	5250.1	14	6	165.130	290.5	47969.
6	6	33.183	175.6	5825.9	15		176.715	296.4	52382.
6	9	35.785	179.9	6436.7	15	6	188.692	302.4	57061.
7		38.485	184.2	7087.	16		201.062	308.4	62008.
7	3	41.283	188.3	7772.7	16	6	213.825	314.2	67183.
7	6	44.179	192.4	8501.8	17		226.981	319.8	72594.
7	9	47.173	196.6	9275.8	17	6	240.529	325.5	78289.
8		50.266	200.6	10083.	18		254.470	331.1	84247.
8	3	53.456	204.5	10934.	19		282.529	342.1	96991.
8	6	56.745	208.5	11832.	20		314.149	352.6	110905.

TABLE 54.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .013$.

Table giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .013$, as in ashlar and well laid brickwork, ordinary metal, earthenware and stoneware pipe, in good condition, but not new, cement and terra cotta pipe not well jointed nor in perfect order, plaster and planed wood in imperfect or inferior condition, and also surfaces of other materials equally rough.

d = diameter in ft.	in.	a = area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	21.20	2.8839	2	11	6.681	93.52	624.82
	6	.196	24.60	4.8216	3		7.068	95.37	674.09
	7	.267	27.87	7.4425	3	1	7.466	97.25	726.05
	8	.349	31.	10.822	3	2	7.875	99.13	780.63
	9	.442	34.	15.029	3	3	8.295	100.9	836.69
	10	.545	36.87	20.095	3	4	8.726	102.8	896.27
	11	.660	39.84	26.296	3	5	9.169	104.4	957.35
1		.785	42.65	33.497	3	6	9.621	106.1	1021.1
1	1	.922	45.22	41.692	3	7	10.084	107.9	1087.7
1	2	1.069	47.85	51.157	3	8	10.559	109.6	1157.2
1	3	1.227	50.42	61.867	3	9	11.044	111.3	1229.7
1	4	1.396	52.90	73.855	3	10	11.541	113.1	1305.3
1	5	1.576	55.44	87.376	3	11	12.048	114.9	1384.1
1	6	1.767	57.80	102.14	4		12.566	116.5	1463.9
1	7	1.969	60.17	118.47	4	1	13.096	118.1	1546.9
1	8	2.182	62.58	136.54	4	2	13.635	119.8	1633.5
1	9	2.405	64.73	155.68	4	3	14.186	121.4	1722.
1	10	2.640	67.07	177.07	4	4	14.748	123.	1813.8
1	11	2.885	69.29	199.90	4	5	15.321	124.6	1908.
2		3.142	71.49	224.63	4	6	15.904	126.2	2007.
2	1	3.409	73.66	251.10	4	7	16.499	127.7	2206.1
2	2	3.687	75.70	279.12	4	8	17.104	129.3	2211.1
2	3	3.976	77.77	309.23	4	9	17.721	130.7	2316.9
2	4	4.276	79.87	341.52	4	10	18.348	132.4	2429.1
2	5	4.587	81.83	375.37	4	11	18.986	134.	2543.9
2	6	4.909	83.82	411.27	5		19.635	135.4	2659.
2	7	5.241	85.95	450.49	5	1	20.205	136.9	2778.7
2	8	5.585	87.89	490.88	5	2	20.966	138.5	2903.5
2	9	5.939	89.71	532.76	5	3	21.648	139.9	3029.4
2	10	6.305	91.68	578.02	5	4	22.340	141.4	3159.

TABLE 54.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .013$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

d = diameter in ft.	in.	a = area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
5	5	23.044	142.9	3292	10	6	86.590	219.4	18996.
5	6	23.758	144.3	3429.	10	9	90.763	222.6.	20205.
5	7	24.484	145.6	3566.	11		95.033	225.9	21464.
5	8	25.220	147.1	3710.	11	3	99.402	229.1	22774.
5	9	25.967	148.6	3859.	11	6	103.869	232.4	24139.
5	10	26.729	150.1	4012.	11	9	108.434	235.4	25533.
5	11	27.494	151.4	4162.	12		113.098	238.6	26981.
6		28.274	152.9	4322.	12	3	117.859	241.7	28484.
6	3	30.680	157.	4816.	12	6	122.719	244.8	30041.
6	6	33.183	161.2	5339.	12	9	127.677	247.8	31633.
6	9	35.785	165.2	5911.	13		132.733	250.9	33301.
7		38.485	169.2	6510.	13	3	137.887	253.8	34996.
7	3	41.283	173.	7142.	13	6	143.139	256.8	36752.
7	6	44.179	176.9	7814.	13	9	148.490	259.7	38561.
7	9	47.173	180.8	8527.	14		153.938	262.6	40432.
8		50.266	184.5	9272.	14	6	165.130	268.4	44322.
8	3	53.456	188.2	10059.	15		176.715	274.	48413.
8	6	56.745	191.9	10889.	15	6	188.692	279.6	52753.
8	9	60.132	195.4	11753.	16		201.062	285.2	57343.
9		63.617	199.1	12663.	16	6	213.825	290.6	62132.
9	3	67.201	202.6	13613.	17		226.981	295.8	67140.
9	6	70.882	205.9	14597.	17	6	240.529	301.	72409.
9	9	74.662	209.3	15629.	18		254.470	306.3	77932.
10		78.540	212.8	16709.	19		282.529	316.6	89759.
10	3	82.516	216.2	17837.	20		314.159	326.5	102559.

TABLE 55.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .015$.

Tables giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel = .015, as in second class or rough faced brickwork; well dressed stonework; foul and slightly tuberculated iron; cement and terra cotta pipes with imperfect joints and in bad order; canvas lining on wooden frames, and also the surfaces of other channels equally rough.

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	17.36	2.3615	2	10	6.305	77.56	488.99
	6	.196	20.21	3.9604	2	11	6.681	79.16	528.85
	7	.267	22.95	6.1268	3		7.068	80.77	570.90
	8	.349	25.56	8.9194	3	1	7.466	82.39	615.14
	9	.442	28.10	12.421	3	2	7.875	84.03	661.77
	10	.545	30.52	16.633	3	3	8.295	85.54	709.56
	11	.660	33.03	21.798	3	4	8.726	87.15	760.44
1		.785	35.40	27.803	3	5	9.169	88.61	812.38
1	1	.922	37.60	34.664	3	6	9.621	90.11	866.91
1	2	1.069	39.85	42.602	3	7	10.084	91.60	923.70
1	3	1.227	42.05	51.600	3	8	10.559	93.11	983.11
1	4	1.396	44.19	61.685	3	9	11.044	94.62	1045.
1	5	1.576	46.36	73.066	3	10	11.541	96.15	1109.6
1	6	1.767	48.38	85.496	3	11	12.048	97.55	1175.2
1	7	1.969	50.40	99.242	4		12.566	99.10	1245.3
1	8	2.182	52.45	114.46	4	1	13.096	100.5	1315.8
1	9	2.405	54.29	130.58	4	2	13.635	102.	1390.8
1	0	2.640	56.29	148.61	4	3	14.186	103.4	1466.7
1	11	2.885	58.20	167.90	4	4	14.748	104.8	1545.7
2		3.142	60.08	188.77	4	5	15.321	106.2	1627.
2	1	3.409	61.95	211.20	4	6	15.904	107.6	1711.4
2	2	3.687	63.72	234.94	4	7	16.499	108.9	1796.5
2	3	3.976	65.51	260.47	4	8	17.104	110.3	1886.8
2	4	4.276	67.32	287.87	4	9	17.721	111.6	1977.7
2	5	4.587	69.02	316.59	4	10	18.348	113.	2074.1
2	6	4.909	70.74	347.28	4	11	18.986	114.4	2172.9
2	7	5.241	72.59	380.46	5		19.635	115.7	2272.7
2	8	5.585	74.27	414.81	5	1	20.295	117.1	2376.7
2	9	5.939	75.98	451.23	5	2	20.966	118.4	2482.

TABLE 55.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .015$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

d = diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	d = diameter in ft.	in.	a = area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
5	3	21.648	119.7	2590.5	12		113.10	206.5	23352.
5	4	22.340	121.	2702.1	12	3	117.86	209.2	24658.
5	5	23.044	122.2	2816.7	12	6	122.72	212.	26012.
5	6	23.758	123.5	2934.8	12	9	127.68	214.6	27399.
5	7	24.484	124.8	3056.4	13		132.73	217.4	28850.
5	8	25.220	126.	3177.3	13	3	137.88	220.	30330.
5	9	25.967	127.3	3305.6	13	6	143.14	222.6	31860.
5	10	26.725	128.6	3436.3	13	9	148.49	225.2	33441.
5	11	27.494	129.7	3566.6	14		153.94	227.8	35073.
6		28.274	131.	3702.3	14	3	159.48	230.	36736.
6	3	30.680	134.6	4130.3	14	6	165.13	232.9	38454.
6	6	33.183	138.3	4588.3	14	9	170.87	235.4	40221.
6	9	35.785	141.8	5074.7	15		176.72	237.9	42040.
7		38.485	145.3	5591.6	15	3	182.65	240.5	43931.
7	3	41.283	148.7	6136.8	15	6	188.69	242.8	45820.
7	6	44.179	152.	6717.	15	9	194.83	245.3	47792.
7	9	47.173	155.5	7333.5	16		201.06	247.8	49823.
8		50.266	158.7	7978.3	16	3	207.40	250.3	51904.
8	3	53.456	162.	8658.8	16	6	213.83	252.7	54056.
8	6	56.745	165.3	9377.9	16	9	220.35	254.9	56171.
8	9	60.132	168.4	10128.	17		226.98	257.2	58387.
9		63.617	171.6	10917.	17	3	233.71	259.7	60700.
9	3	67.201	174.7	11740.	17	6	240.53	261.9	62999.
9	6	70.882	177.7	12594.	17	9	247.45	264.4	65428.
9	9	74.662	180.7	13489.	18		254.47	266.6	67839.
10		78.540	183.7	14426.	18	3	261.59	268.9	70346.
10	3	82.516	186.7	15406.	18	6	268.80	271.3	72916.
10	6	86.590	189.5	16412.	18	9	276.12	273.5	75507.
10	9	90.763	192.4	17462.	19		283.53	275.8	78201.
11		95.033	195.2	18555.	19	3	291.04	278.	80216.
11	3	99.402	198.1	19694.	19	6	298.65	280.2	83686
11	6	103.87	201.	20879.	19	9	306.36	282.4	86526.
11	9	108.43	203.7	22093.	20		314.16	284.6	89423.

TABLE 56.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .017$.

Tables giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .017$, as for brickwork, ashlar and stoneware in an inferior condition; tuberculated iron pipes; rubble in cement or plaster in good order; fine gravel, well rammed, $\frac{1}{2}$ to $\frac{3}{4}$ inches diameter; and generally the materials mentioned with $n = .013$ when in bad order and condition, and the surfaces of other channels equally rough.

Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Diameter in ft.	in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	5	.136	14.55	1.979	6	6	33.183	120.8	4010.
	6	.196	16.98	3.329	6	9	35.785	124.	4437.9
	7	.267	19.33	5.162	7		38.485	127.1	4893.
	8	.349	21.59	7.535	7	3	41.283	130.1	5373.3
	9	.442	23.76	10.50	7	6	44.179	133.2	5884.2
	10	.545	25.84	14.08	7	9	47.173	136.2	6427.9
	11	.660	28.	18.48	8		50.266	139.2	6995.3
1		.785	30.05	23.60	8	3	53.456	142.	7594.1
1	1	.922	31.95	29.46	8	6	56.745	145.	8226.3
1	2	1.069	33.90	36.24	8	9	60.132	147.8	8886.4
1	3	1.227	35.80	43.93	9		63.617	150.6	9580 7
1	4	1.396	37.65	52.56	9	3	67.201	153.4	10307.
1	5	1.576	39.55	62.33	9	6	70.882	156.	11061.
1	6	1.767	41.31	72.99	9	9	74.662	158.7	11851.
1	7	1.969	43.07	84.81	10		78.540	161.4	12678.
1	8	2.182	44.88	97.92	10	3	82.516	164.1	13544.
1	9	2.405	46.49	111.8	10	6	86.590	166.7	14434.
1	10	2.640	48.25	127.3	10	9	90.763	169.3	15364.
1	11	2.885	49.92	144.	11		95.033	171.9	16333
2		3.142	51.57	164.	11	3	99.402	174.5	17343.
2	3	3.976	56.32	223.9	11	6	103.869	177.1	18395.
2	6	4.909	60.98	299.3	11	9	108.434	179.5	19468.
2	9	5.939	65.47	388.8	12		113.098	182.	20584.
3		7.068	69.80	493.3	12	6	122.719	186.9	22938.
3	3	8.295	74.	613.9	13		132.733	191.7	25451.
3	6	9.621	78.04	750.8	13	6	143.139	196.4	28117.
3	9	11.044	82.04	906.	14		153.938	201.1	30965.
4		12.566	86.	1080.7	14	6	165.130	205.7	33975.
4	3	14.186	89.79	1273.8	15		176.715	210.2	37147.
4	6	15.904	93.51	1487.3	15	6	188.692	214.7	40510.
4	9	17.721	97.05	1719.9	16		201.062	219.2	44073.
5		19.635	100.6	1977.	16	6	213.825	223.5	47784.
5	3	21.648	104.2	2255.8	17		226.981	227.6	51669.
5	6	23.758	107.6	2557.2	17	6	240.529	231.8	55762.
5	9	25.967	111.	2882.1	18		254.470	236.	60067.
6		28.274	114.3	3232.5	19		282.529	244.4	69301.
6	3	30.680	117.5	3606.8	20		314.159	252.3	79259.

TABLE 57.

Circular Pipes, Conduits, Sewers, etc., flowing full. Based on Kutter's formula with $n = .020$.

Table giving the values of a and r, and also the values of factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of $n = .020$, as in rubble in cement in an inferior condition; coarse rubble rough set in a normal condition; coarse rubble set dry; ruined brickwork and masonry; coarse gravel well rammed, from 1 to 1½ inch diameter; canals with beds and banks of very firm, regular gravel, carefully trimmed and rammed in defective places; rough rubble, with bed partially covered with silt and mud; rectangular wooden troughs, with battens on the inside two inches apart; trimmed earth in perfect order, and surfaces of other materials equally rough.

diameter in ft.	diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	diameter in ft.	diameter in in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
	6	.196	13.56	2.658	6		28.274	95.85	2710.2
	9	.442	19.10	8.442	6	6	33.183	101.4	3365.6
1		.785	24.30	19.07	7		38.485	106.8	4111.4
1	3	1.227	29.08	35.68	7	6	44.179	112.1	4951.
1	6	1.767	33.66	59.49	8		50.266	117.2	5891.5
1	9	2.405	38.01	91.42	9		63.617	127.2	8092.1
2		3.142	42.29	132.9	10		78.540	136.6	10731.
2	3	3.976	46.31	184.1	11		95.033	145.8	13856.
2	6	4.909	50.20	246.4	12		113.098	154.5	17479.
2	9	5.939	54.01	320.8	13		132.733	163.	21639.
3		7.068	57.71	407.9	14		153.938	171.3	26365.
3	3	8.295	61.23	507.9	15		176.715	179.1	31660.
3	6	9.621	64.72	622.7	16		201.062	186.9	37583.
3	9	11.044	68.13	752.4	17		226.981	194.4	44119.
4		12.566	71.50	898.5	18		254.470	201.6	51312.
4	6	15.904	77.93	1239.4	19		283.529	208.9	59238.
5		19.635	84.10	1651.2	20		314.159	215.9	67837.
5	6	23.758	90.12	2140.8					

TABLE 58.

Giving the value of the hydraulic mean depth r, for egg-shaped sewers flowing full depth, two-thirds full depth and one-third full depth.

Let D = transverse diameter, that is, diameter of top of sewer, then
Hydraulic mean depth of sewer flowing full depth $= D \times 0.2897$.
Hydraulic mean depth of sewer flowing $\frac{2}{3}$ full depth $= D \times 0.3157$.
Hydraulic mean depth of sewer flowing $\frac{1}{3}$ full depth $= D \times 0.2066$.

Size of Sewer	r = hydraulic mean depth in feet			Size of Sewer	r = hydraulic mean depth in feet		
ft. in. ft. in.	Full Depth	$\frac{2}{3}$ Full Depth	$\frac{1}{3}$ Full Depth	ft. in. ft. in.	Full Depth	$\frac{2}{3}$ Full Depth	$\frac{1}{3}$ Full Depth
1 ×1 6	.2897	.316	.207	5 2× 7 9	1.497	1.631	1.068
1 2×1 9	.3380	.368	.241	5 4× 8	1.545	1.684	1.102
1 4×2	.3864	.421	.276	5 6× 8 3	1.593	1.736	1.136
1 6×2 3	.4345	.474	.310	5 8× 8 6	1.642	1.789	1.171
1 8×2 6	.4828	.526	.344	5 10× 8 9	1.690	1.842	1.205
1 10×2 9	.5311	.579	.379	6 × 9	1.738	1.894	1.240
2 ×3	.5794	.631	.413	6 2× 9 3	1.787	1.947	1.274
2 2×3 3	.6277	.684	.448	6 4× 9 6	1.835	1.999	1.309
2 4×3 6	.6760	.737	.482	6 6× 9 9	1.883	2.052	1.343
2 6×3 9	.7242	.789	.517	6 8×10	1.931	2.095	1.377
2 8×4	.7725	.842	.551	6 10×10 3	1.980	2.157	1.412
2 10×4 3	.8208	.894	.585	7 ×10 6	2.028	2.210	1.446
3 ×4 6	.8691	.947	.620	7 4×11	2.124	2.315	1.515
3 2×4 9	.9174	1.000	.654	7 8×11 6	2.221	2.420	1.584
3 4×5	.9657	1.052	.689	8 ×12	2.318	2.526	1.653
3 6×5 3	1.014	1.105	.723	8 4×12 6	2.414	2.631	1.722
3 8×5 6	1.062	1.158	.758	8 8×13	2.511	2.736	1.791
3 10×5 9	1.111	1.210	.792	9 ×13 6	2.607	2.841	1.859
4 ×6	1.159	1.263	.826	9 4×14	2.704	2.947	1.928
4 2×6 3	1.207	1.315	.861	9 8×14 6	2.800	3.052	1.997
4 4×6 6	1.255	1.368	.895	10 ×15	2.897	3.157	2.066
4 6×6 9	1.304	1.421	.930	10 6×15 9	3.042	3.315	2.169
4 8×7	1.352	1.473	.964	11 ×16 6	3.187	3.473	2.273
4 10×7 3	1.400	1.526	.999	12 ×18	3.476	3.788	2.479
5 ×7 6	1.449	1.579	1.033				

TABLE 59.

Egg-shaped Sewers flowing full depth. Based on Kutter's formula with $n = .011$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only when the value of $n = .011$ as in plaster in cement with one-third sand in good condition; also for iron, cement, and terra cotta pipes, well jointed and in best order, and also the surfaces of other materials equally rough.

The egg-shaped sewer referred to has a vertical diameter equal to 1.5 times the greatest transverse diameter D, that is, the diameter of the top of sewer.

Area of egg-shaped sewer flowing full depth $= D^2 \times 1.148525$.

Perimeter of egg-shaped sewer flowing full depth $= D \times 3.9649$.

Hydraulic mean depth of egg-shaped sewer flowing full depth $= D \times 0.2897$.

Size of sewer ft. in. ft. in.	a=area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer ft. in. ft. in.	a = area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	1.1485	58.8	67.5	5 2× 7 9	30.660	182.7	5602.
1 2×1 9	1.5632	65.9	102.9	5 4× 8	32.669	186.5	6093.5
1 4×2	2.0417	72.7	148.4	5 6× 8 3	34.743	190.2	6607.5
1 6×2 3	2.5841	78.9	204.	5 8× 8 6	36.880	193.8	7150.2
1 8×2 6	3.1903	85.2	272.	5 10× 8 9	39.081	197.6	7722.4
1 10×2 9	3.8602	91.1	351.7	6 × 9	41.347	201.	8312.7
2 ×3	4.5941	96.8	444.7	6 2× 9 3	43.676	204.7	8940.8
2 2×3 3	5.3914	102.3	551.7	6 4× 9 6	46.068	208.	9582.1
2 4×3 6	6.2529	107.7	673.3	6 6× 9 9	48.525	211.5	10263.
2 6×3 9	7.1783	112.9	810.6	6 8×10	51.046	215.	10976.
2 8×4	8.1674	118.	964.1	6 10×10 3	53.629	218.3	11709.
2 10×4 3	9.2198	123.	1134.3	7 ×10 6	56.278	221.6	12473.
3 ×4 6	10.377	127.7	1325.1	7 4×11	61.764	228.1	14087.
3 2×4 9	11.517	132.5	1526.	7 8×11 6	67.508	234.6	15835.
3 4×5	12.761	137.1	1749.9	8 ×12	73.506	240.8	17704.
3 6×5 3	14.069	141.7	1993.3	8 4×12 6	79.758	247.1	19713.
3 8×5 6	15.442	146.1	2255.9	8 8×13	86.268	253.3	21853.
3 10×5 9	16.877	150.4	2538.4	9 ×13 6	93.031	259.2	24119.
4 ×6	18.376	154.7	2843.9	9 4×14	100.049	264.9	26509.
4 2×6 3	19.940	159.	3170.9	9 8×14 6	107.324	270.7	29051.
4 4×6 6	21.566	162.9	3514.4	10 ×15	114.853	276.5	31754.
4 6×6 9	23.258	167.	3885.8	10 6×15 9	126.625	284.7	36058.
4 8×7	25.013	171.	4279.1	11 ×16 6	138.972	292.9	40707.
4 10×7 3	26.830	174.9	4694.3	12 ×18	165.388	308.7	51051.
5 ×7 6	28.713	179.	5140.6				

TABLE 60.

Egg-shaped Sewers flowing two-thirds full depth. Based on Kutter's formula with $n = .011$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer. Area of egg-shaped sewer flowing two-thirds full depth $= D^2 \times 0.755825$. Perimeter of egg-shaped sewer flowing two-thirds full depth $= D \times 2.3941$. Hydraulic mean depth of egg-shaped sewer flowing two-thirds full depth $= D \times 0.3157$.

Size of Sewer ft. in. ft. in.	a=area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of Sewer ft. in. ft. in.	a=area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.7558	62.71	47.40	5 2× 7 9	20.176	193.1	3896.2
1 2×1 9	1.0287	70.26	72.27	5 4× 8	21.498	197.2	4239.5
1 4×2	1.3436	77.27	103.8	5 6× 8 3	22.864	201.	4596.7
1 6×2 3	1.7005	84.04	142.9	5 8× 8 6	24.269	204.9	4972.8
1 8×2 6	2.0994	90.63	190.3	5 10× 8 9	25.718	208.6	5364.3
1 10×2 9	2.5402	96.79	245.9	6 × 9	27.210	212.3	5776.3
2 ×3	3.0232	102.9	311.2	6 2× 9 3	28.742	216.	6208.8
2 2×3 3	3.5480	108.6	385.4	6 4× 9 6	30.317	219.7	6660.6
2 4×3 6	4.1149	114.2	469.9	6 6× 9 9	31.933	223.4	7133.6
2 6×3 9	4.7237	119.9	566.6	6 8×10	33.592	226.9	7622.3
2 8×4	5.3746	125.2	672.9	6 10×10 3	35.292	230.4	8132.3
2 10×4 3	6.0674	130.3	790.6	7 ×10 6	37.035	234.	8670.0
3 ×4 6	6.8022	135.3	920.5	7 4×11	40.646	240.8	9789.8
3 2×4 9	7.5790	140.4	1064.1	7 8×11 6	44.426	247.5	10988.
3 4×5	8.3970	145.2	1219.3	8 ×12	48.372	254.1	12293.
3 6×5 3	9.2585	149.8	1387.5	8 4×12 6	52.487	260.6	13679.
3 8×5 6	10.161	154.6	1570.8	8 8×13	56.771	266.9	15154.
3 10×5 9	11.106	159.2	1767.7	9 ×13 6	61.222	273.3	16731.
4 ×6	12.093	163.7	1979.6	9 4×14	65.840	279.4	18397.
4 2×6 3	13.122	168.1	2205.5	9 8×14 6	70.628	285.3	20154.
4 4×6 6	14.192	172.5	2448.	10 ×15	75.582	291.3	22018.
4 6×6 9	15.305	176.7	2705.3	10 6×15 9	83.330	300.1	25007.
4 8×7	16.460	181.1	2981.6	11 ×16 6	91.455	308.7	28233.
4 10×7 3	17.656	185.	3266.2	12 ×18	108.838	325.1	35387.
5 ×7 6	18.895	189.	3571.8				

TABLE 61.

Egg-shaped Sewers flowing one-third full depth. Based on Kutter's formula with $n = .011$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}.$$

The egg-shaped sewer referred to has a vertical diameter 1.5 the greatest transverse diameter, D, that is, the diameter of the top of the sewer.
Area of egg-shaped sewer flowing one-third full depth $= D^2 \times 0.284$.
Perimeter of egg-shaped sewer flowing one-third full depth $= D \times 1.3747$.
Hydraulic mean depth of egg-shaped sewer flowing one-third full depth $= D \times 0.2066$.

Size of Sewer ft. in. ft. in.	a=area in square feet	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of Sewer ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.2840	45.72	12.98	5 2× 7 9	7.5812	146.5	1110.6
1 2×1 9	.3865	51.39	19.89	5 4× 8	8.0782	149.7	1209.1
1 4×2	.5049	56.74	28.65	5 6× 8 3	8.5910	152.7	1311.8
1 6×2 3	.6390	61.89	39.55	5 8× 8 6	9.1196	155.7	1420.3
1 8×2 6	.7889	66.90	52.78	5 10× 8 9	9.6639	158.8	1534.4
1 10×2 9	.9545	71.58	68.36	6 × 9	10.224	161.6	1652.4
2 ×3	1.1360	76.26	86.63	6 2× 9 3	10.800	164.6	1778.1
2 2×3 3	1.3332	80.71	107.6	6 4× 9 6	11.391	167.5	1908.1
2 4×3 6	1.5462	85.28	131.8	6 6× 9 9	11.999	170.4	2044.3
2 6×3 9	1.7750	89.42	158.7	6 8×10	12.622	173.3	2187.
2 8×4	2.0195	93.42	188.7	6 10×10 3	13.261	176.	2334.7
2 10×4 3	2.2799	97.50	222.3	7 ×10 6	13.916	178.9	2489.4
3 ×4 6	2.5560	101.6	259.8	7 4×11	15.273	184.2	2813.5
3 2×4 9	2.8479	105.4	300.2	7 8×11 6	16.693	189.4	3161.9
3 4×5	3.1556	109.1	344.5	8 ×12	18.176	194.8	3541.7
3 6×5 3	3.4790	112.7	392.3	8 4×12 6	19.722	199.9	3942.3
3 8×5 6	3.8182	116.4	444.4	8 8×13	21.331	204.9	4370.8
3 10×5 9	4.1732	120.1	501.1	9 ×13 6	23.004	209.9	4829.6
4 ×6	4.5440	123.6	561.5	9 4×14	24.739	214.8	5314.8
4 2×6 3	4.9306	127.	626.3	9 8×14 6	26.538	219.5	5825.3
4 4×6 6	5.3329	130.3	694.9	10 ×15	28.400	224.2	6366.4
4 6×6 9	5.7510	133.6	768.6	10 6×15 9	31.311	231.2	7239.6
4 8×7	6.1849	137.	847.4	11 ×16 6	34.364	237.9	8176.
4 10×7 3	6.6346	140.4	931.5	12 ×18	40.892	251.3	10277.
5 ×7 6	7.100	143.3	1017.8				

TABLE 62.

Egg-shaped Sewers flowing full depth. Based on Kutter's formula with $n = .013$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The factors are to be used only where the value of n, that is the co-efficient of roughness of lining of channel $= .013$ as in ashlar and well laid brickwork; ordinary metal; earthenware and stoneware pipe, in good condition but not new; cement and terra cotta pipe not well jointed nor in perfect order, and also plaster and planed wood in imperfect or inferior condition and generally the materials mentioned with $n = .010$ when in imperfect or inferior condition and also the surfaces of other materials equally rough.

The egg-shaped sewer referred to has a vertical diameter equal to 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer.

Area of egg-shaped sewer flowing full depth $= D^2 \times 1.148525$.

Perimeter of egg-shaped sewer flowing two-thirds full depth $= D \times 3.9649$.

Hydraulic mean depth of egg-shaped sewer flowing one-third full depth $= D \times 0.2897$.

Size of Sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of Sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	1.148	47.58	54.653	5 2× 7 9	30.66	152.5	4677.4
1 2×1 9	1.563	53.46	83.585	5 4× 8	32.669	155.8	5031.4
1 4×2	2.041	59.19	120.83	5 6× 8 3	34.743	159.	5523.7
1 6×2 3	2.584	64.44	166.53	5 8× 8 6	36.88	162.1	5980.5
1 8×2 6	3.19	69.74	222.48	5 10× 8 9	39.081	165.3	6462.4
1 10×2 9	3.86	74.68	288.27	6 × 9	41.347	168.3	6960.1
2 ×3	4.594	79.42	364.85	6 2× 9 3	43.676	171.5	7490.3
2 2×3 3	5.391	84.12	453.56	6 4× 9 6	46.068	174.3	8032.2
2 4×3 6	6.253	88.64	554.29	6 6× 9 9	48.525	177.4	8607.6
2 6×3 9	7.178	93.06	667.99	6 8×10	51.046	180.4	9210.5
2 8×4	8.167	97.40	795.52	6 10×10 3	53.629	183.3	9830.4
2 10×4 3	9.22	101.6	937.06	7 ×10 6	56.278	186.1	10476.
3 ×4 6	10.337	105.6	1092.2	7 4×11	61.764	191.7	11841.
3 2×4 9	11.517	109.7	1264.1	7 8×11 6	67.508	197.3	13322.
3 4×5	12.761	113.7	1451.6	8 ×12	73.506	202.7	14903.
3 6×5 3	14.069	117.6	1654.5	8 4×12 6	79.758	208.1	16601.
3 8×5 6	15.442	121.4	1874.5	8 8×13	86.268	213.4	18413.
3 10×5 9	16.877	125.1	2110.8	9 ×13 6	93.03	218.5	20331.
4 ×6	18.376	128.8	2366.6	9 4×14	100.049	223.4	22356.
4 2×6 3	19.94	132.4	2639.8	9 8×14 6	107.324	228.4	24514.
4 4×6 6	21.566	135.7	2927.5	10 ×15	114.853	223.4	26808.
4 6×6 9	23.258	139.3	3239.6	10 6×15 9	126.625	240.6	30471.
4 8×7	25.013	142.7	3569.6	11 ×16 6	138.972	247.7	34431.
4 10×7 3	26.83	146.	3917.	12 ×18	165.388	261.4	13237.
5 ×7 6	28.713	149.4	4291.2				

TABLE 63.

Egg-shaped Sewers flowing two-thirds full. Bas with $n = .013$.

Giving the value of a, and also the values of th for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r}$$

The egg-shaped sewer referred to has a vertical greatest transverse diameter, D, that is, the diame Area of egg-shaped sewer flowing two-thirds full d Perimeter of egg-shaped sewer flowing two-thirds Hydraulic mean depth of egg-shaped sewer flowi $= D \times 0.3157$.

Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer. ft. in. ft. in
1 ×1 6	.756	50.83	38.42	5 2× 7 9
1 2×1 9	1.029	57.12	58.76	5 4× 8
1 4×2	1.344	63.	84.65	5 6× 8 3
1 6×2 3	1.701	68.7	116.82	5 8× 8 6
1 8×2 6	2.099	74.24	155.86	5 10× 8 9
1 10×2 9	2.540	79.42	201.74	6 × 9
2 ×3	3.023	84.59	255.73	6 2× 9 3
2 2×3 3	3.548	89.4	317.19	6 4× 9 6
2 4×3 6	4.115	94.14	387.38	6 6× 9 9
2 6×3 9	4.724	98.97	467.52	6 8×10
2 8×4	5.375	103.5	556.2	6 10×10 3
2 10×4 3	6.067	107.8	654.45	7 ×10 6
3 ×4 6	6.802	112.1	762.85	7 4×11
3 2×4 9	7.579	116.5	882.95	7 8×11 6
3 4×5	8.398	120.6	1012.7	8 ×12
3 6×5 3	9.259	124.6	1153.4	8 4×12 6
3 8×5 6	10.161	128.6	1307.	8 8×13
3 10×5 9	11.106	132.5	1472.1	9 ×13 6
4 ×6	12.093	136.4	1649.3	9 4×14
4 2×6 3	13.123	140.1	1838.5	9 8×14 6
4 4×6 6	14.192	143.8	2041.5	10 ×15
4 6×6 9	15.305	147.5	2257.1	10 6×15 9
4 8×7	16.46	151.1	2486.8	11 ×16 6
4 10×7 3	17.656	154.5	2728.3	12 ×18
5 ×7 6	18.895	158.	2985.4	

TABLE 64.

Egg-shaped Sewers flowing one-third full depth. Based on Kutter's formula with $n = .013$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of the sewer.

Area of egg-shaped sewer flowing one-third full depth $= D^2 \times .284$.

Perimeter of egg-shaped sewer flowing one-third full depth $= D \times 1.3747$.

Hydraulic mean depth of egg-shaped sewer flowing one-third full depth $= D \times .2066$.

Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer. ft. in ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.284	36.74	10.436	5 2× 7 9	7.581	121.7	922.69
1 2×1 9	.387	41.43	16.015	5 4× 8	8.078	124.4	1005.1
1 4×2	.505	45.87	23.162	5 6× 8 3	8.591	127.	1091.1
1 6×2 3	.639	50.14	32.044	5 8× 8 6	9.120	129.6	1181.9
1 8×2 6	.789	54.31	42.845	5 10× 8 9	9.664	132.2	1277.8
1 10×2 9	.955	58.22	55.573	6 × 9	10.224	134.6	1376.4
2 ×3	1.136	62.14	70.598	6 2× 9 3	10.8	137.2	1481.7
2 2×3 3	1.333	65.89	87.853	6 4× 9 6	11.391	139.6	1590.3
2 4×3 6	1.546	69.74	107.84	6 6× 9 9	12.999	142.	1704.6
2 6×3 9	1.776	73.22	129.97	6 8×10	12.622	144.5	1824.
2 8×4	2.020	76.59	154.67	6 10×10 3	13.261	147.	1949.2
2 10×4 3	2.280	80.02	182.44	7 ×10 6	13.916	149.3	2077.6
3 ×4 6	2.556	83.51	213.46	7 4×11	15.273	153.8	2349.9
3 2×4 9	2.848	86.70	246.91	7 8×11 6	16.693	158.3	2643.
3 4×5	3.156	89.85	283.55	8 ×12	18.176	163.	2962.7
3 6×5 3	3.479	92.90	323.22	8 4×12 6	19.722	167.3	3300.4
3 8×5 6	3.818	96.	366.53	8 8×13	21.332	171.7	3662.
3 10×5 9	4.173	99.13	413.68	9 ×13 6	23.004	176.	4049.6
4 ×6	4.544	102.1	463.9	9 4×14	24.739	180.2	4459.6
4 2×6 3	4.931	105.	517.91	9 8×14 6	26.538	184.3	4891.3
4 4×6 6	5.333	107.8	575.22	10 ×15	28.4	188.3	5348.7
4 6×6 9	5.751	110.7	636.6	10 6×15 9	31.311	194.4	6088.
4 8×7	6.189	113.6	702.5	11 ×16 6	34.364	200.2	6880.4
4 10×7 3	6.635	116.5	772.9	12 ×18	40.892	211.7	8658.
5 ×7 6	7.100	119.	845.				

TABLE 65.

Egg-shaped Sewer flowing full depth. Based on Kutter's formula with $n = .015$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the co-efficient of roughness of lining of channel = .015, as in second-class or rough faced brickwork; well-dressed stonework; foul and slightly tuber-culated iron; cement and terra cotta pipes, with imperfect joints and in bad order, and canvas lining on wooden frames, and also the surfaces of other materials equally rough.

The egg-shaped sewer referred to has a vertical diameter equal to 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer.

Area of egg-shaped sewer flowing full depth = $D^2 \times 1.148525$.
Perimeter of egg-shaped sewer flowing full depth = $D \times 3.9649$.
Hydraulic mean depth of egg-shaped sewer flowing full depth = $D \times 0.2897$.

Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	1.148	39.62	45.528	5 2× 7 9	30.660	130.7	4007
1 2×1 9	1.563	44.66	69.804	5 4× 8	32.669	133.6	4364
1 4×2	2.041	49.57	101.17	5 6× 8 3	34.743	136.4	4738
1 6×2 3	2.584	54.08	139.74	5 8× 8 6	36.880	139.2	5131
1 8×2 6	3.190	58.64	187.06	5 10× 8 9	39.081	142.	5548
1 10×2 9	3.860	62.83	242.52	6 × 9	41.347	144.6	5980
2 ×3	4.594	66.93	307.48	6 2× 9 3	43.676	147.3	6435.
2 2×3 3	5.391	71.01	382.81	6 4× 9 6	46.068	149.8	6902
2 4×3 6	6.253	74.93	468.54	6 6× 9 9	48.525	152.5	7399
2 6×3 9	7.178	78.76	565.34	6 8×10	51.046	155.2	7920
2 8×4	8.167	82.44	673.29	6 10×10 3	53.629	157.7	8547
2 10×4 3	9.220	86.21	794.86	7 ×10 6	56.278	160.2	9015
3 ×4 6	10.337	89.70	927.23	7 4×11	61.764	165.	10192
3 2×4 9	11.517	93.25	1074.	7 8×11 6	67.508	170.1	11482
3 4×5	12.761	96.73	1234.4	8 ×12	73.506	174.8	12852
3 6×5 3	14.069	100.1	1407.6	8 4×12 6	79.758	179.6	14327
3 8×5 6	15.442	103.4	1596.7	8 8×13	86.268	184.3	15898
3 10×5 9	16.877	106.6	1799.1	9 ×13 6	93.030	188.8	17563
4 ×6	18.376	109.9	2019.5	9 4×14	100.049	193.1	19323
4 2×6 3	19.940	113.	2254.	9 8×14 6	107.324	197.5	21198
4 4×6 6	21.566	116.	2501.4	10 ×15	114.853	201.9	23191
4 6×6 9	23.258	119.1	2770.	10 6×15 9	126.625	208.3	26376
4 8×7	25.013	122.1	3053.8	11 ×16 6	138.972	214.6	29822
4 10×7 3	26.830	125.	3353.	12 ×18	165.388	226.8	37502
5 ×7 6	28.713	128.	3675.6				

TABLE 66.

Egg-shaped Sewers flowing two-thirds full depth. Based on Kutter's formula with $n = .015$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer. Area of egg-shaped sewer flowing two-thirds full depth $= D^2 \times 0.755825$. Perimeter of egg-shaped sewer flowing two-thirds full depth $= D \times 2.3941$. Hydraulic mean depth of egg-shaped sewer flowing two-thirds full depth $= D \times .03157$.

Size of sewer ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer ft. in. ft. in	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.756	42.40	32.048	5 2× 7 9	20.177	138.6	2795.9
1 2×1 9	1.029	47.80	49.181	5 4× 8	21.498	141.7	3045.5
1 4×2	1.344	52.82	70.993	5 6× 8 3	22.863	144.6	3305.3
1 6×2 3	1.701	57.68	98.115	5 8× 8 6	24.270	147.5	3578.9
1 8×2 6	2 099	62.46	131.10	5 10× 8 9	25.718	150.3	3864.8
1 10×2 9	2.540	66.94	170.02	6 × 9	27.210	153.1	4165.3
2 ×3	3.023	71.42	216.54	6 2× 9 3	28.743	155.9	4481.6
2 2×3 3	3.548	75.59	268.19	6 4× 9 6	30.317	158.7	4811.9
2 4×3 6	4.115	79.69	327.93	6 6× 9 9	31.933	161.5	5158.5
2 6×3 9	4.724	83.90	396.32	6 8×10	33.592	164.2	5516.6
2 8×4	5.375	87.82	472.01	6 10×10 3	35.292	166.9	5891.
2 10×4 3	6 067	91.60	555.74	7 ×10 6	37.035	169.6	6283.5
3 ×4 6	6.802	95.33	648.40	7 4×11	40.646	174.8	7106.8
3 2×4 9	7.579	99.10	751.08	7 8×11 6	44.426	179.9	7993.
3 4×5	8.398	102.7	862.41	8 ×12	48.373	184.9	8944.
3 6×5 3	9.259	106.2	983.24	8 4×12 6	52.487	189.8	9964.1
3 8×5 6	10.161	109.7	1115.1	8 8×13	56.771	194.6	11050.
3 10×5 9	11.106	113.2	1256.1	9 ×13 6	61.222	199.5	12213.
4 ×6	12.093	116.5	1409.4	9 4×14	65.840	204.2	13444.
4 2×6 3	13.123	119.8	1572.1	9 8×14 6	70.628	208.7	14743.
4 4×6 6	14.192	123.1	1746.9	10 ×15	75.583	213.3	16125.
4 6×6 9	15.305	126.3	1932.7	10 6×15 9	83.330	220.1	18342.
4 8×7	16.460	129.4	2130.5	11 ×16 6	91.455	226.8	20738.
4 10×7 3	17.656	132.5	2338.6	12 ×18	108.84	239.4	26060.
5 ×7 6	18.895	135 5	2560.3				

TABLE 63.

Egg-shaped Sewers flowing two-thirds full. Based on Kutter's formula with $n = .013$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer. Area of egg-shaped sewer flowing two-thirds full depth $= D^2 \times 0.755825$. Perimeter of egg-shaped sewer flowing two-thirds full depth $= D \times 2.3941$. Hydraulic mean depth of egg-shaped sewer flowing two-thirds full depth $= D \times 0.3157$.

Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.756	50.83	38.42	5 2× 7 9	20.177	161.5	3258.4
1 2×1 9	1.029	57.12	58.76	5 4× 8	21.498	165.	3547.8
1 4×2	1.344	63.	84.65	5 6× 8 3	22.863	168.3	3848.8
1 6×2 3	1.701	68.7	116.82	5 8× 8 6	24.270	171.7	4166.3
1 8×2 6	2.099	74.24	155.86	5 10× 8 9	25.718	174.8	4496.8
1 10×2 9	2.540	79.42	201.74	6 × 9	27.21	178.	4844.9
2 ×3	3.023	84.59	255.73	6 2× 9 3	28.743	181.3	5210.9
2 2×3 3	3.548	89.4	317.19	6 4× 9 6	30.317	184.5	5603.7
2 4×3 6	4.115	94.14	387.38	6 6× 9 9	31.933	187.7	5992.9
2 6×3 9	4.724	98.97	467.52	6 8×10	33.592	190.7	6406.4
2 8×4	5.375	103.5	556.2	6 10×10 3	35.292	193.7	6837.9
2 10×4 3	6.067	107.8	654.45	7 ×10 6	37.035	196.8	7289.2
3 ×4 6	6.802	112.1	762.85	7 4×11	40.646	202.7	8240.8
3 2×4 9	7.579	116.5	882.95	7 8×11 6	44.426	208.5	9262.3
3 4×5	8.398	120.6	1012.7	8 ×12	48.373	214.1	10358.
3 6×5 3	9.259	124.6	1153.4	8 4×12 6	52.487	219.7	11532.
3 8×5 6	10.161	128.6	1307.	8 8×13	56.771	225.1	12783.
3 10×5 9	11.106	132.5	1472.1	9 ×13 6	61.222	230.6	14122.
4 ×6	12.093	136.4	1649.3	9 4×14	65.84	236.	15537.
4 2×6 3	13.123	140.1	1838.5	9 8×14 6	70.628	241.1	17032.
4 4×6 6	14.192	143.8	2041.5	10 ×15	75.583	246.3	18621.
4 6×6 9	15.305	147.5	2257.1	10 6×15 9	83.33	254.	21165.
4 8×7	16.46	151.1	2486.8	11 ×16 6	91.455	261.4	23909.
4 10×7 3	17.656	154.5	2728.3	12 ×18	108.84	275.7	30008.
5 ×7 6	18.895	158.	2985.4				

TABLE 64.

Egg-shaped Sewers flowing one-third full depth. Based on Kutter's formula with $n = .013$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of the sewer.

Area of egg-shaped sewer flowing one-third full depth $= D^2 \times .284$.

Perimeter of egg-shaped sewer flowing one-third full depth $= D \times 1.3747$.

Hydraulic mean depth of egg-shaped sewer flowing one-third full depth $= D \times .2066$.

Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer. ft. in ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.284	36.74	10.436	5 2× 7 9	7.581	121.7	922.69
1 2×1 9	.387	41.43	16.015	5 4× 8	8.078	124.4	1005.1
1 4×2	.505	45.87	23.162	5 6× 8 3	8.591	127.	1091.1
1 6×2 3	.639	50.14	32.044	5 8× 8 6	9.120	129.6	1181.9
1 8×2 6	.789	54.31	42.845	5 10× 8 9	9.664	132.2	1277.8
1 10×2 9	.955	58.22	55.573	6 × 9	10.224	134.6	1376.4
2 ×3	1.136	62.14	70.598	6 2× 9 3	10.8	137.2	1481.7
2 2×3 3	1.333	65.89	87.853	6 4× 9 6	11.391	139.6	1590.3
2 4×3 6	1.546	69.74	107.84	6 6× 9 9	12.999	142.	1704.6
2 6×3 9	1.776	73.22	129.97	6 8×10	12.622	144.5	1824.
2 8×4	2.020	76.59	154.67	6 10×10 3	13.261	147.	1949.2
2 10×4 3	2.280	80.02	182.44	7 ×10 6	13.916	149.3	2077.6
3 ×4 6	2.556	83.51	213.46	7 4×11	15.273	153.8	2349.9
3 2×4 9	2.848	86.70	246.91	7 8×11 6	16.693	158.3	2643.
3 4×5	3.156	89.85	283.55	8 ×12	18.176	163.	2962.7
3 6×5 3	3.479	92.90	323.22	8 4×12 6	19.722	167.3	3300.4
3 8×5 6	3.818	96.	366.53	8 8×13	21.332	171.7	3662.
3 10×5 9	4.173	99.13	413.68	9 ×13 6	23.004	176.	4049.6
4 ×6	4.544	102.1	463.9	9 4×14	24.739	180.2	4459.6
4 2×6 3	4.931	105.	517.91	9 8×14 6	26.538	184.3	4891.3
4 4×6 6	5.333	107.8	575.22	10 ×15	28.4	188.3	5348.7
4 6×6 9	5.751	110.7	636.6	10 6×15 9	31.311	194.4	6088.
4 8×7	6.189	113.6	702.5	11 ×16 6	34.364	200.2	6880.4
4 10×7 3	6.635	116.5	772.9	12 ×18	40.892	211.7	8658.
5 ×7 6	7.100	119.	845.				

TABLE 65.

Egg-shaped Sewer flowing full depth. Based on Kutter's formula with $n = .015$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

These factors are to be used only where the value of n, that is the coefficient of roughness of lining of channel $= .015$, as in second-class or rough faced brickwork; well-dressed stonework; foul and slightly tuberculated iron; cement and terra cotta pipes, with imperfect joints and in bad order, and canvas lining on wooden frames, and also the surfaces of other materials equally rough.

The egg-shaped sewer referred to has a vertical diameter equal to 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer.

Area of egg-shaped sewer flowing full depth $= D^2 \times 1.148525$.
Perimeter of egg-shaped sewer flowing full depth $= D \times 3.9649$.
Hydraulic mean depth of egg-shaped sewer flowing full depth $= D \times 0.2897$.

Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer. ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	1.148	39.62	45.528	5 2× 7 9	30.660	130.7	4007.9
1 2×1 9	1.563	44.66	69 804	5 4× 8	32.669	133.6	4364.9
1 4×2	2.041	49.57	101.17	5 6× 8 3	34.743	136.4	4738.
1 6×2 3	2.584	54.08	139.74	5 8× 8 6	36.880	139.2	5131.7
1 8×2 6	3.190	58.64	187.06	5 10× 8 9	39.081	142.	5548.
1 10×2 9	3.860	62.83	242.52	6 × 9	41.347	144.6	5980.3
2 ×3	4.594	66.93	307.48	6 2× 9 3	43.676	147.3	6435.1
2 2×3 3	5.391	71.01	382.81	6 4× 9 6	46.068	149.8	6902.6
2 4×3 6	6.253	74.93	468.54	6 6× 9 9	48.525	152.5	7399.3
2 6×3 9	7.178	78.76	565.34	6 8×10	51.046	155.2	7920.6
2 8×4	8.167	82.44	673.29	6 10×10 3	53.629	157.7	8547.1
2 10×4 3	9.220	86.21	794.86	7 ×10 6	56.278	160.2	9015.7
3 ×4 6	10.337	89.70	927.23	7 4×11	61.764	165.	10192.
3 2×4 9	11.517	93.25	1074.	7 8×11 6	67.508	170.1	11482.
3 4×5	12.761	96.73	1234.4	8 ×12	73.506	174.8	12852.
3 6×5 3	14.069	100.1	1407.6	8 4×12 6	79.758	179.6	14327
3 8×5 6	15.442	103.4	1596.7	8 8×13	86.268	184.3	15898.
3 10×5 9	16.877	106.6	1799.1	9 ×13 6	93.030	188.8	17563.
4 ×6	18.376	109.9	2019.5	9 4×14	100.049	193.1	19323.
4 2×6 3	19.940	113.	2254.	9 8×14 6	107.324	197.5	21198.
4 4×6 6	21.566	116.	2501.4	10 ×15	114.853	201.9	23191.
4 6×6 9	23.258	119.1	2770.	10 6×15 9	126.625	208.3	26376.
4 8×7	25.013	122.1	3053 8	11 ×16 6	138.972	214.6	29822.
4 10×7 3	26.830	125.	3353.	12 ×18	165.388	226.8	37502.
5 ×7 6	28.713	128.	3675.6				

TABLE 66.

Egg-shaped Sewers flowing two-thirds full depth. Based on Kutter's formula with $n = .015$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of sewer.
Area of egg-shaped sewer flowing two-thirds full depth $= D^2 \times 0.755825$.
Perimeter of egg-shaped sewer flowing two-thirds full depth $= D \times 2.3941$.
Hydraulic mean depth of egg-shaped sewer flowing two-thirds full depth $= D \times .03157$.

Size of sewer ft. in. ft. in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer ft. in. ft. in.	a = area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 × 1 6	.756	42.40	32.048	5 2 × 7 9	20.177	138.6	2795.9
1 2 × 1 9	1.029	47.80	49.181	5 4 × 8	21.498	141.7	3045.5
1 4 × 2	1.344	52.82	70.993	5 6 × 8 3	22.863	144.6	3305.3
1 6 × 2 3	1.701	57.68	98.115	5 8 × 8 6	24.270	147.5	3578.9
1 8 × 2 6	2.099	62.46	131.10	5 10 × 8 9	25.718	150.3	3864.8
1 10 × 2 9	2.540	66.94	170.02	6 × 9	27.210	153.1	4165.3
2 × 3	3.023	71.42	216.54	6 2 × 9 3	28.743	155.9	4481.6
2 2 × 3 3	3.548	75.59	268.19	6 4 × 9 6	30.317	158.7	4811.9
2 4 × 3 6	4.115	79.69	327.93	6 6 × 9 9	31.933	161.5	5158.5
2 6 × 3 9	4.724	83.90	396.32	6 8 × 10	33.592	164.2	5516.6
2 8 × 4	5.375	87.82	472.01	6 10 × 10 3	35.292	166.9	5891.
2 10 × 4 3	6.067	91.60	555.74	7 × 10 6	37.035	169.6	6283.5
3 × 4 6	6.802	95.33	648.40	7 4 × 11	40.646	174.8	7106.8
3 2 × 4 9	7.579	99.10	751.08	7 8 × 11 6	44.426	179.9	7993.
3 4 × 5	8.398	102.7	862.41	8 × 12	48.373	184.9	8944.
3 6 × 5 3	9.259	106.2	983.24	8 4 × 12 6	52.487	189.8	9964.1
3 8 × 5 6	10.161	109.7	1115.1	8 8 × 13	56.771	194.6	11050.
3 10 × 5 9	11.106	113.2	1256.1	9 × 13 6	61.222	199.5	12213.
4 × 6	12.093	116.5	1409.4	9 4 × 14	65.840	204.2	13444.
4 2 × 6 3	13.123	119.8	1572.1	9 8 × 14 6	70.628	208.7	14743.
4 4 × 6 6	14.192	123.1	1746.9	10 × 15	75.583	213.3	16125.
4 6 × 6 9	15.305	126.3	1932.7	10 6 × 15 9	83.330	220.1	18342.
4 8 × 7	16.460	129.4	2130.5	11 × 16 6	91.455	226.8	20738.
4 10 × 7 3	17.656	132.5	2338.6	12 × 18	108.84	239.4	26060.
5 × 7 6	18.895	135.5	2560.3				

TABLE 67.

Egg-shaped Sewers flowing one-third full depth. Based on Kutter's formula with $n = .015$.

Giving the value of a, and also the values of the factors $c\sqrt{r}$ and $ac\sqrt{r}$ for use in the formulæ:—

$$v = c\sqrt{r} \times \sqrt{s} \text{ and } Q = ac\sqrt{r} \times \sqrt{s}$$

The egg-shaped sewer referred to has a vertical diameter 1.5 times the greatest transverse diameter, D, that is, the diameter of the top of the sewer.

Area of egg-shaped sewer flowing one-third full depth $= D^2 \times .284$.
Perimeter of egg-shaped sewer flowing one-third full depth $= D \times 1.3747$.
Hydraulic mean depth of egg-shaped sewer flowing one-third full depth $= D \times .2066$.

Size of sewer ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$	Size of sewer ft. in. ft. in.	a=area in square feet.	For velocity $c\sqrt{r}$	For discharge $ac\sqrt{r}$
1 ×1 6	.284	30.41	8.637	5 2× 7 9	7.581	103.7	785.86
1 2×1 9	.387	34.38	13.303	5 4× 8	8.078	106.1	856.67
1 4×2	.505	38.16	19.269	5 6× 8 3	8.591	108.3	930.54
1 6×2 3	.639	42.23	26.986	5 8× 8 6	9.120	110.6	1008.7
1 8×2 6	.789	45.39	35.815	5 10× 8 9	9.664	112.9	1091.
1 10×2 9	.955	48.74	46.546	6 × 9	10.224	115.	1175.8
2 ×3	1 136	52.09	59.173	6 2× 9 3	10.800	117.3	1266.4
2 2×3 3	1.333	55.29	73.696	6 4× 9 6	11.391	119.4	1359.8
2 4×3 6	1.546	58.58	90.568	6 6× 9 9	12.999	121.5	1458.1
2 6×3 9	1.776	61.58	109.37	6 8×10	12.622	123.7	1561.
2 8×4	2.020	64.49	130.26	6 10×10 3	13.261	125.8	1668.8
2 10×4 3	2.280	67.46	153.80	7 ×10 6	13.916	127.9	1779.4
3 ×4 6	2.556	70.48	180.14	7 4×11	15.273	131.9	2014.1
3 2×4 9	2.848	73.24	208.98	7 8×11 6	16.693	135.8	2266.7
3 4×5	3.156	75.98	239.79	8 ×12	18.176	139.9	2542.7
3 6×5 3	3.479	78.63	273.54	8 4×12 6	19.722	143.7	2833.8
3 8×5 6	3.818	81.31	310.44	8 8×13	21.332	147.5	3146.2
3 10×5 9	4.173	84.03	350.67	9 ×13 6	23.004	151.3	3480.7
4 ×6	4.544	86.61	393.55	9 4×14	24.739	155.	3834.7
4 2×6 3	4.931	88.98	438.75	9 8×14 6	26.538	158.6	4208.4
4 4×6 6	5.333	91.60	488.50	10 ×15	28.400	162.1	4604.7
3 6×6 9	5.751	94.08	541.04	10 6×15 9	31.311	167.5	5245.3
4 8×7	6.189	96.57	597.29	11 ×16 6	34.364	172.6	5932.1
4 10×7 3	6.635	99.10	657.53	12 ×18	40.892	183.1	7489.
5 ×7 6	7.100	101.3	719.27				

TABLE 68.

Giving velocities and discharges of Circular Pipes, Sewers and Conduits, based on Kutter's formula, with $n = .013$.

d = diameter.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	$d = 5''$		$d = 6''$		$d = 7''$		$d = 8''$		$d = 9''$	
	v	Q	v	Q	v	Q	v	Q	v	Q
40	3.35	.456	3.89	.762	4.40	1.17	4.90	1.71	5.37	2.37
70	2.53	.344	2.94	.576	3.33	.889	3.7	1.29	4.06	1.79
100	2.12	.288	2.46	.482	2.79	.744	3.1	1.08	3.40	1.50
200	1.50	.204	1.74	.341	1.97	.526	2.19	.765	2.4	1.06
300	1.22	.166	1.42	.278	1.61	.430	1.79	.624	1.96	.868
400	1.06	.144	1.23	.241	1.39	.372	1.55	.54	1.7	.75
500	1.01	.137	1.17	.230	1.33	.355	1.48	.516	1.62	.717
600	.865	.118	1.	.197	1.14	.304	1.26	.441	1.39	.613

Slope 1 in	$d = 10''$		$d = 11''$		$d = 1'\ 0''$		$d = 1'\ 1''$		$d = 1'\ 2''$	
	v	Q	v	Q	v	Q	v	Q	v	Q
60	4.76	2.59	5.14	3.39	5.5	4.32	5.84	5.38	6.18	6.6
80	4.12	2.24	4.45	2.94	4.77	3.74	5.05	4.66	5.35	5.72
100	3.68	1.	3.98	2.63	4.26	3.35	4.52	4.16	4.78	5.15
200	2.61	1.42	2.82	1.86	3.01	2.37	3.2	2.95	3.38	3.62
300	2.13	1.16	2.3	1.52	2.46	1.93	2.61	2.4	2.76	2.95
400	1.84	.5	1.99	1.31	2.13	1.67	2.26	2.08	2.39	2.57
500	1.65	.9	1.78	1.17	1.91	1.5	2.02	1.86	2.14	2.29
600	1.5	.82	1.62	1.07	1.74	1.37	1.84	1.70	1.95	2.09

Slope 1 in	$d = 1'\ 3''$		$d = 1'\ 4''$		$d = 1'\ 6''$		$d = 1'\ 8''$		$d = 1'\ 10''$	
	v	Q	v	Q	v	Q	v	Q	v	Q
100	5.04	6.18	5.29	7.38	5.78	10.21	6.25	13.65	6.70	17.71
200	3.56	4.37	3.74	5.22	4.09	7.22	4.43	9.65	4.74	12.52
300	2.91	3.57	3.05	4.26	3.34	5.89	3.61	7.88	3.87	10.22
400	2.52	3.09	2.64	3.69	2.89	5.10	3.12	6.82	3.35	8.85
500	2.25	2.77	2.36	3.30	2.58	4.56	2.8	6.1	3.	7.92
600	2.06	2.52	2.16	3.01	2.36	4.17	2.56	5.57	2.74	7.23
700	1.90	2.34	2.	2.79	2.18	3.86	2.37	5.16	2.53	6.69
800	1.78	2.19	1.87	2.61	2.04	3.61	2.21	4.83	2.37	6.26

Slope 1 in	$d = 2'\ 0''$		$d = 2'\ 2''$		$d = 2'\ 4''$		$d = 2'\ 6''$		$d = 2'\ 8''$	
	v	Q	v	Q	v	Q	v	Q	v	Q
200	5.05	15.88	5.35	19.73	5.65	24.15	5.92	29.08	6.21	34.71
400	3.57	11.23	3.78	13.96	3.99	17.07	4.19	20.56	4.39	24.54
600	2.92	9.17	3.09	11.39	3.26	13.94	3.42	16.79	3.59	20.04
800	2.53	7.94	2.67	9.87	2.82	12.07	2.96	14.54	3.11	17.35
1000	2.26	7.1	2.39	8.82	2.52	10.8	2.65	13.	2.78	15.52
1250	2.02	6.35	2.14	7.89	2.26	9.66	2.37	11.63	2.48	13.88
1500	1.84	5.8	1.95	7.2	2.06	8.82	2.16	10.62	2.27	12.67
1800	1.68	5.29	1.78	6.58	1.88	8.05	1.97	9.69	2.07	11.57

TABLE 68.

Giving velocities and discharges of Circular Pipes, Sewers and Conduits, based on Kutter's formula, with $n = .013$.

d = diameter.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	$d = 2' 10''$		$d = 3' 0''$		$d = 3' 2''$		$d = 3' 4''$		$d = 3' 6''$	
	v	Q	v	Q	v	Q	v	Q	v	Q
500	4.10	25.84	4.26	30.14	4.43	34.90	4.59	40.08	4.74	45.66
750	3.34	21.10	3.48	24.61	3.61	28.50	3.75	32.72	3.87	37.28
1000	2.89	18·27	3.01	21.31	3.13	24.68	3.25	28.34	3.35	32.28
1250	2.59	16.34	2.69	19.06	2.80	22.07	2.90	25.35	3.	28.87
1500	2.36	14.92	2.46	17.40	2.55	20.15	2.65	23.14	2.73	26.36
1750	2.19	13.81	2.28	16.11	2.36	18.66	2.45	21.42	2.53	24.40
2000	2.05	12.92	2.13	15.07	2.21	17.45	2.29	20.04	2.37	22.83
2640	1.78	11.24	1.85	13.12	1.92	15.19	2.	17.44	2.06	19.87
	$d = 3' 8''$		$d = 3' 10''$		$d = 4' 0''$		$d = 4' 6''$		$d = 5' 0''$	
500	4.90	51.74	5.06	58.36	5.21	65.47	5.64	89.75	6.05	118.9
750	4.	42.52	4.13	47.65	4.25	53.46	4.61	73.28	4.94	97.09
1000	3.46	36.59	3.58	41.27	3.68	46.3	3.99	63.47	4.28	84.08
1250	3.1	32.72	3.2	36.91	3.29	41.41	3.57	56.76	3.83	75.21
1500	2.83	29.87	2.92	33.69	3.01	37.8	3.26	51.82	3.49	68.65
1750	2.62	27.66	2.7	31.2	2.78	34.5	3.01	47.97	3.24	63.56
2000	2.45	25.87	2.53	29.18	2.61	32.74	2.82	44.88	3.02	59.46
2640	2.13	22.52	2.2	25.4	2.27	28.49	2.46	39.06	2.63	51.75
	$d = 5' 6''$		$d = 6' 0''$		$d = 6' 6''$		$d = 7' 0''$		$d = 7' 6''$	
750	5.27	125.2	5.58	157.8	5.88	195.	6 18	237.7	6.46	285.3
1000	4.56	108.4	4.83	136.7	5.1	168.8	5.35	205.9	5.59	247.1
1500	3.72	88.54	3.95	111.6	4.16	137.9	4.37	168.1	4.57	201.7
2000	3.22	76.67	3.42	96.66	3.60	119.4	3.78	145.6	3.95	174.7
2500	2.88	68.58	3.06	86.45	3.22	106.8	3.38	130.2	3.53	156.3
3000	2.63	62.6	2.79	78.92	2.94	97.49	3.09	118.8	3.23	142.6
3500	2.44	57.96	2.58	73.07	2.72	90.26	2.86	110.	2.99	132.1
4000	2.28	54.21	2.42	68.35	2.55	84.43	2.67	102.9	2.8	123.5
	$d = 8' 0''$		$d = 8' 6''$		$d = 9' 0''$		$d = 9' 6''$		$d = 10' 0''$	
1500	4.76	239.4	4.95	281.1	5.14	327.	5.31	376.9	5.49	431.4
2000	4.12	207.3	4.29	243.5	4.45	283.1	4.6	326.4	4.76	373.6
2500	3.69	195.4	3.84	217.8	3.98	253.3	4.12	291.9	4.25	334.1
3000	3.37	169.3	3.50	198.8	3.63	231.2	3.76	266.5	3.88	305.
3500	3.12	156.7	3.24	184.	3.36	214.	3.48	246.7	3.6	282.4
4000	2.92	146.6	3.03	172.2	3.15	200.2	3.25	230.8	3.36	264.2
4500	2.75	138.2	2.86	162.3	2.97	188.7	3.07	217.6	3.17	249.1
5000	2.61	131.1	2.71	154.	2.81	179.1	2.91	206.4	3.01	236.3

TABLE 69.

Giving velocities and discharges of Egg-Shaped Sewers, based on Kutter's formula, with $n = .013$. Flowing full depth. Flowing $\frac{2}{3}$ full depth. Flowing $\frac{1}{3}$ full depth.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	Full Depth		$\frac{2}{3}$ Full Depth		$\frac{1}{3}$ Full Depth	
	v	Q	v	Q	v	Q
	Size of Sewer 2′ 9″ x 3′ 0″					
100	7.94	36.48	8.46	25.57	6.21	7.06
200	5.61	25.8	5.98	18.08	4.39	4.99
300	4.58	21.06	4.88	14.76	3.59	4.07
500	3.55	16.31	3.78	11.43	2.78	3.16
700	3.	13.79	3.2	9.66	2.35	2.67
1000	2.51	11.54	2.67	8.08	1.96	2.23
1200	2.29	10.53	2.44	7.38	1.79	2.04
1500	2.05	9.42	2.18	6.6	1.6	1.82
	Size of Sewer 2′ 2″ x 3′ 3″					
100	8.41	45.35	8.94	31.72	6.59	8.78
200	5.95	32.07	6.32	22.43	4.66	6.21
300	4.85	26.19	5.16	18.31	3.80	5.07
500	4.01	21.64	4.26	15.14	3.14	4.19
700	3.18	17.14	3.38	11.99	2.49	3.32
1000	2.66	14.34	2.83	10.03	2.08	2.78
1200	2.43	13.09	2.58	9.15	1.9	2.53
1500	2.17	12.71	2.31	8.19	1.7	2.26
	Size of Sewer 2′ 4″ x 3′ 6″					
150	7.24	45.26	7.68	31.63	5.69	8.8
300	5.12	32.	5.43	22.37	4.02	6.22
600	3.62	22.63	3.84	15.81	2.84	4.4
1000	2.8	17.53	2.97	12.25	2.2	3.41
1250	2.51	15.68	2.66	10.96	1.97	3.05
1500	2.29	14.31	2.43	10.	1.8	2.78
1750	2.12	13.25	2.25	9.26	1.67	2.58
2000	1.98	12.39	2.1	8.66	1.56	2.41
	Size of Sewer 2′ 6″ x 3′ 9″					
300	5.37	38.57	5.71	26.99	4.2	7.5
600	3.8	27.27	4.04	19.08	2.98	5.31
1000	2.94	21.12	3.13	14.78	2.31	4.11
1250	2.63	18.89	2.8	13.22	2.06	3.68
1500	2.4	17.25	2.55	12.07	1.88	3.36
1750	2.22	15.97	2.37	11.17	1.74	3.11
2000	2.08	14.94	2.21	10.45	1.63	2.91
2640	1.81	13.	1.93	9.1	1.42	2.53

TABLE 68.

Giving velocities and discharges of Circular Pipes, Sewers and Conduits, based on Kutter's formula, with $n = .013$.

d = diameter.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	$d = 2' 10''$		$d = 3' 0''$		$d = 3' 2''$		$d = 3' 4''$		$d = 3' 6''$	
	v	Q	v	Q	v	Q	v	Q	v	Q
500	4.10	25.84	4.26	30.14	4.43	34.90	4.59	40.08	4.74	45.66
750	3.34	21.10	3.48	24.61	3.61	28.50	3.75	32.72	3.87	37.28
1000	2.89	18·27	3.01	21.31	3.13	24.68	3.25	28.34	3.35	32.28
1250	2.59	16.34	2.69	19.06	2.80	22.07	2.90	25.35	3.	28.87
1500	2.36	14.92	2.46	17.40	2.55	20.15	2.65	23.14	2.73	26.36
1750	2.19	13.81	2.28	16.11	2.36	18.66	2.45	21.42	2.53	24.40
2000	2.05	12.92	2.13	15.07	2.21	17.45	2.29	20.04	2.37	22.83
2640	1.78	11.24	1.85	13.12	1.92	15.19	2.	17.44	2.06	19.87
	$d = 3' 8''$		$d = 3' 10''$		$d = 4' 0''$		$d = 4' 6''$		$d = 5' 0''$	
500	4.90	51.74	5.06	58.36	5.21	65.47	5.64	89.75	6.05	118.9
750	4.	42.52	4.13	47.65	4.25	53.46	4.61	73.28	4.94	97.09
1000	3.46	36.59	3.58	41.27	3.68	46.3	3.99	63.47	4.28	84.08
1250	3.1	32.72	3.2	36.91	3.29	41.41	3.57	56.76	3.83	75.21
1500	2.83	29.87	2.92	33.69	3.01	37.8	3.26	51.82	3.49	68.65
1750	2.62	27.66	2.7	31.2	2.78	34.5	3.01	47.97	3.24	63.56
2000	2.45	25.87	2.53	29.18	2.61	32.74	2.82	44.88	3.02	59.46
2640	2.13	22.52	2.2	25.4	2.27	28.49	2.46	39.06	2.63	51.75
	$d = 5' 6''$		$d = 6' 0''$		$d = 6' 6''$		$d = 7' 0''$		$d = 7' 6''$	
750	5.27	125.2	5.58	157.8	5.88	195.	6.18	237.7	6.46	285.3
1000	4.56	108.4	4.83	136.7	5.1	168.8	5.35	205.9	5.59	247.1
1500	3.72	88.54	3.95	111.6	4.16	137.9	4.37	168.1	4.57	201.7
2000	3.22	76.67	3.42	96.66	3.60	119.4	3.78	145.6	3.95	174.7
2500	2.88	68.58	3.06	86.45	3.22	106.8	3.38	130.2	3.53	156.3
3000	2.63	62.6	2.79	78.92	2.94	97.49	3.09	118.8	3.23	142.6
3500	2.44	57.96	2.58	73.07	2.72	90.26	2.86	110.	2.99	132.1
4000	2.28	54.21	2.42	68.35	2.55	84.43	2.67	102.9	2.8	123.5
	$d = 8' 0''$		$d = 8' 6''$		$d = 9' 0''$		$d = 9' 6''$		$d = 10' 0''$	
1500	4.76	239.4	4.95	281.1	5.14	327.	5.31	376.9	5.49	431.4
2000	4.12	207.3	4.29	243.5	4.45	283.1	4.6	326.4	4.76	373.6
2500	3.69	195.4	3.84	217.8	3.98	253.3	4.12	291.9	4.25	334.1
3000	3.37	169.3	3.50	198.8	3.63	231.2	3.76	266.5	3.88	305.
3500	3.12	156.7	3.24	184.	3.36	214.	3.48	246.7	3.6	282.4
4000	2.92	146.6	3.03	172.2	3.15	200.2	3.25	230.8	3.36	264.2
4500	2.75	138.2	2.86	162.3	2.97	188.7	3.07	217.6	3.17	249.1
5000	2.61	131.1	2.71	154.	2.81	179.1	2.91	206.4	3.01	236.3

TABLE 69.

Giving velocities and discharges of Egg-Shaped Sewers, based on Kutter's formula, with $n = .013$. Flowing full depth. Flowing $\frac{2}{3}$ full depth. Flowing $\frac{1}{3}$ full depth.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	Full Depth v	Full Depth Q	$\frac{2}{3}$ Full Depth v	$\frac{2}{3}$ Full Depth Q	$\frac{1}{3}$ Full Depth v	$\frac{1}{3}$ Full Depth Q
	Size of Sewer 2′ 9″ x 3′ 0″					
100	7.94	36.48	8.46	25.57	6.21	7.06
200	5.61	25.8	5.98	18.08	4.39	4.99
300	4.58	21.06	4.88	14.76	3.59	4.07
500	3.55	16.31	3.78	11.43	2.78	3.16
700	3.	13.79	3.2	9.66	2.35	2.67
1000	2.51	11.54	2.67	8.08	1.96	2.23
1200	2.29	10.53	2.44	7.38	1.79	2.04
1500	2.05	9.42	2.18	6.6	1.6	1.82
	Size of Sewer 2′ 2″ x 3′ 3″					
100	8.41	45.35	8.94	31.72	6.59	8.78
200	5.95	32.07	6.32	22.43	4.66	6.21
300	4.85	26.19	5.16	18.31	3.80	5.07
500	4.01	21.64	4.26	15.14	3.14	4.19
700	3.18	17.14	3.38	11.99	2.49	3.32
1000	2.66	14.34	2.83	10.03	2.08	2.78
1200	2.43	13.09	2.58	9.15	1.9	2.53
1500	2.17	12.71	2.31	8.19	1.7	2.26
	Size of Sewer 2′ 4″ x 3′ 6″					
150	7.24	45.26	7.68	31.63	5.69	8.8
300	5.12	32.	5.43	22.37	4.02	6.22
600	3.62	22.63	3.84	15.81	2.84	4.4
1000	2.8	17.53	2.97	12.25	2.2	3.41
1250	2.51	15.68	2.66	10.96	1.97	3.05
1500	2.29	14.31	2.43	10.	1.8	2.78
1750	2.12	13.25	2.25	9.26	1.67	2.58
2000	1.98	12.39	2.1	8.66	1.56	2.41
	Size of Sewer 2′ 6″ x 3′ 9″					
300	5.37	38.57	5.71	26.99	4.2	7.5
600	3.8	27.27	4.04	19.08	2.98	5.31
1000	2.94	21.12	3.13	14.78	2.31	4.11
1250	2.63	18.89	2.8	13.22	2.06	3.68
1500	2.4	17.25	2.55	12.07	1.88	3.36
1750	2.22	15.97	2.37	11.17	1.74	3.11
2000	2.08	14.94	2.21	10.45	1.63	2.91
2640	1.81	13.	1.93	9.1	1.42	2.53

TABLE 69.

Giving velocities and discharges of Egg-Shaped Sewers, based on Kutter's formula, with $n = .013$. Flowing full depth. Flowing ⅔ full depth. Flowing ⅓ full depth.

v = velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	Full Depth v	Full Depth Q	⅔ Full Depth v	⅔ Full Depth Q	⅓ Full Depth v	⅓ Full Depth Q
	Size of Sewer 2′ 8″ x 4′ 0″					
500	4.35	35.57	4.62	24.87	3.42	6.91
750	3.55	29.04	3.77	20.30	2.79	5.64
1000	3.08	25.15	3.27	17.58	2.42	4.89
1250	2.75	22.49	2.92	15.73	2.16	4.37
1500	2.51	20.53	2.67	14.36	1.97	3.99
1750	2.32	19.01	2.47	13.29	1.83	3.69
2000	2.17	17.78	2.31	12.43	1.71	3.45
2640	1.89	15.48	2.01	10.82	1.49	3.01
	Size of Sewer 2′ 10″ x 4′ 3″					
500	4.54	41.90	4.82	29.26	3.57	8.15
750	3.70	34.21	3.93	23.89	2.92	6.66
1000	3.21	29.63	3.41	20.69	2.52	5.76
1250	2.87	26.50	3.05	18.50	2.26	5.15
1500	2.62	24.19	2.78	16.89	2.06	4.70
1750	2.42	22.39	2.57	15.64	1.91	4.36
2000	2.27	20.95	2.41	14.63	1.78	4.07
2640	1.97	18.23	2.10	12.73	1.55	3.55
	Size of Sewer 3′ 0″ x 4′ 6″					
500	4.72	48.83	5.01	34.11	3.73	9.54
750	3.85	39.87	4.09	27.85	3.04	7.79
1000	3.33	34.53	3.54	24.12	2.64	6.74
1250	2.98	30.88	3.17	21.57	2.36	6.03
1500	2.72	28.19	2.89	19.69	2.15	5.50
1750	2.52	26.10	2.67	18.23	1.99	5.10
2000	2.36	24.41	2.50	17.05	1.86	4.77
2640	2.05	21.25	2.18	14.84	1.62	4.15
	Size of Sewer 3′ 2″ x 4′ 9″					
500	4.90	56.52	5.20	39.48	3.87	11.04
750	4.	46.15	4.25	32.24	3.16	9.01
1000	3.46	39.97	3.68	27.92	2.74	7.80
1250	3.10	35.75	3.29	24.97	2.45	6.98
1500	2.83	32.63	3.	22.79	2.23	6.37
1750	2.62	30.21	2.78	21.10	2.07	5.90
2000	2.45	28.26	2.60	19.74	1.93	5.52
2640	2.13	24.60	2.26	17.18	1.68	4.80

TABLE 69.

Giving velocities and discharges of Egg-Shaped Sewers, based on Kutter's formula, with $n = .013$. Flowing full depth. Flowing $\frac{2}{3}$ full depth. Flowing $\frac{1}{3}$ full depth.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	Full Depth v	Full Depth Q	$\frac{2}{3}$ Full Depth v	$\frac{2}{3}$ Full Depth Q	$\frac{1}{3}$ Full Depth v	$\frac{1}{3}$ Full Depth Q
	Size of Sewer 3′ 4″ x 5′ 0″					
500	5.08	64.89	5.39	45.25	4.01	12.67
750	4.15	52.98	4.40	36.95	3.27	10.35
1000	3.59	45.88	3.81	32.	2.83	8.96
1250	3.21	41.	3.41	28.62	2.53	8.01
1500	2.93	37.46	3.11	26.13	2.32	7.32
1750	2.72	34.68	2.88	24.19	2.14	6.77
2000	2.54	32.44	2.69	22.63	2.01	6.34
2640	2.21	28.24	2.34	19.69	1.74	5.51
	Size of Sewer 3′ 6″ x 5′ 3″					
500	5.26	73.97	5.57	51.56	4.15	14.45
750	4.29	60.39	4.55	42.10	3.39	11.80
1000	3.72	52.30	3.94	36.46	2.94	10.22
1250	3.32	46.78	3.52	32.61	2.62	9.14
1500	3.03	42.70	3.21	29.77	2.40	8.34
1750	2.81	39.53	2.98	27.56	2.22	7.72
2000	2.63	36.98	2.78	25.78	2.08	7.22
2640	2.29	32.19	2.42	22.44	1.81	6.29
	Size of Sewer 3′ 8″ x 5′ 6″					
500	5.43	83.81	5.75	58.45	4.29	16.39
750	4.43	68.43	4.69	47.72	3.50	13.38
1000	3.84	59.26	4.07	41.33	3.03	11.59
1250	3.43	53.	3.64	36.97	2.71	10.37
1500	3.13	48.39	3.32	33.74	2.48	9.46
1750	2.9	44.8	3.07	31.24	2.29	8.76
2000	2.71	41.9	2.87	29.22	2.14	8.19
2640	2.36	36.47	2.50	25.44	1.87	7.13
	Size of Sewer 3′ 10″ x 5′ 9″					
750	4.56	77.08	4.84	53.75	3.62	15.11
1000	3.95	66.76	4.19	46.55	3.13	13.08
1250	3.53	59.71	4.03	41.63	2.8	11.7
1500	3.23	54.51	3.42	38.	2.56	10.68
1750	2.99	50.46	3.17	35.19	2.37	9.89
2000	2.79	47.2	2.96	32.91	2.22	9.25
2640	2.43	41.09	2.58	28.65	1.93	8.05
3000	2.28	38.54	2.42	26.87	1.81	7.55

TABLE 69.

Giving velocities and discharges of Egg-Shaped Sewers, based on Kutter's formula, with $n = .013$. Flowing full depth. Flowing ⅔ full depth. Flowing ⅓ full depth.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	Full Depth v	Full Depth Q	⅔ Full Depth v	⅔ Full Depth Q	⅓ Full Depth v	⅓ Full Depth Q
	Size of Sewer 4′ 0″ x 6′ 0″					
1000	4.07	74.82	4.31	52.14	3.22	14.66
1250	3.64	66.91	3.85	46.64	2.88	13.12
1500	3.32	61.09	3.52	42.57	2.63	11.97
1750	3.07	56.66	3.26	39.41	2.44	11.08
2000	2.88	52.90	3.05	36.87	2.28	10.37
2640	2.50	46.04	2.65	32.09	1.98	9.02
3000	2.35	43.19	2.49	30.10	1.86	8.46
3500	2.17	39.99	2.30	27.87	1.72	7.84
	Size of Sewer 4′ 2″ x 6′ 3″					
1000	4.18	83.48	4.43	58.12	3.32	16.37
1250	3.74	74.66	3.96	51.98	2.96	14.64
1500	3.41	68.16	3.61	47.45	2.71	13.37
1750	3.16	63.10	3.34	43.93	2.51	12.38
2000	2.96	59.03	3.13	41.09	2.34	11.58
2640	2.57	51.38	2.72	35.77	2.04	10.07
3000	2.41	48.19	2.55	33.55	1.91	9.45
3500	2.29	44.62	2.36	31.06	1.77	8.75
	Size of Sewer 4′ 4″ x 6′ 6″					
1250	3.84	82.79	4.07	57.73	3.05	16.27
1500	3.5	75.57	3.71	52.7	2.78	14.85
1750	3.24	69.97	3.44	48.79	2.58	13.45
2000	3.03	65.45	3.21	45.64	2.41	12.86
2640	2.64	56.97	2.8	39.72	2.1	11.19
3000	2.48	53.44	2.62	37.26	1.97	10.5
3500	2.29	49.47	2.43	34.5	1.82	9.72
4000	2.14	46.28	2.27	32.27	1.7	9.09
	Size of Sewer 4′ 6″ x 6′ 9″					
1250	3.94	91.61	4.17	63.84	3.13	18.01
1500	3.6	83.63	3.81	58.27	2.85	16.44
1750	3.33	77.43	3.52	53.95	2.65	15.22
2000	3.11	72.42	3.3	50.47	2.47	14.24
2640	2.71	63.04	2.87	43.93	2.15	12.39
3000	2.54	59.13	2.69	41.21	2.02	11.62
3500	2.35	54.75	2.49	38.15	1.87	10.76
4000	2.2	51.21	2.33	35.68	1.75	10.07

TABLE 69.

Giving velocities and discharges of Egg-Shaped Sewers, based on Kutter's formula, with $n = .013$. Flowing full depth. Flowing $\frac{2}{3}$ full depth. Flowing $\frac{1}{3}$ full depth.

v = mean velocity in feet per second.

Q = discharge in cubic feet per second.

Slope 1 in	Full Depth		$\frac{2}{3}$ Full Depth		$\frac{1}{3}$ Full Depth	
	v	Q	v	Q	v	Q
	Size of Sewer 4′ 8″ x 7′ 0″					
1250	4.04	101.	4.27	70.34	3.21	19.87
1500	3.68	92.17	3.9	64.21	2.93	18.14
1750	3.41	85.34	3.61	59.45	2.71	16.79
2000	3.19	79.82	3.38	55.61	2.54	15.7
2640	2.78	69.48	2.94	48.4	2.21	13.67
3000	2.60	65.18	2.76	45.4	2.07	12.83
3500	2.41	60.34	2.55	42.04	1.92	11.87
4000	2.26	56.44	2.39	39.31	1.79	11.11
	Size of Sewer 4′ 10″ x 7′ 3″					
1250	4.13	110.8	4.37	77.16	3.29	21.86
1500	3.77	101.1	3.99	70.43	3.01	19.96
1750	3.49	93.63	3.69	65.21	2.78	18.48
2000	3.26	87.59	3.45	61.	2.60	17.28
2640	2.84	76.24	3.01	53.09	2.27	15.04
3000	2.66	71.51	2.82	49.8	2.13	14.11
3500	2.47	66.21	2.61	46.11	1.97	13.06
4000	2.31	61.93	2.44	43.13	1.84	12.22
	Size of Sewer 5′ 0″ x 7′ 6″					
1500	3.86	110.8	4.08	77.07	3.07	21.82
1750	3.57	102.6	3.78	71.35	2.84	20.2
2000	3.34	95.95	3.53	66.75	2.66	18.9
2640	2.91	83.51	3.07	58.1	2.32	16.45
3000	2.73	78.34	2.88	54.5	2.17	15.43
3500	2.52	72.53	2.67	50.45	2.01	14.28
4000	2.36	67.84	2.5	47.2	1.88	13.36
5000	2.11	60.68	2.23	42.21	1.68	11.95
	Size of Sewer 5′ 4″ x 8′ 0″					
1500	4.02	131.4	4.26	91.61	3.21	25.95
1750	3.72	121.7	3.94	84.81	2.97	24.02
2000	3.48	113.8	3.69	79.33	2.78	22.47
2640	3.03	99.1	3.21	69.05	2.42	19.56
3000	2.84	92.95	3.01	64.77	2.27	18.35
3500	2.63	86.05	2.79	60.	2.1	17.
4000	2.46	80.49	2.61	56.1	1.97	15.89
5000	2.2	72.	2.33	50.18	1.76	14.21

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